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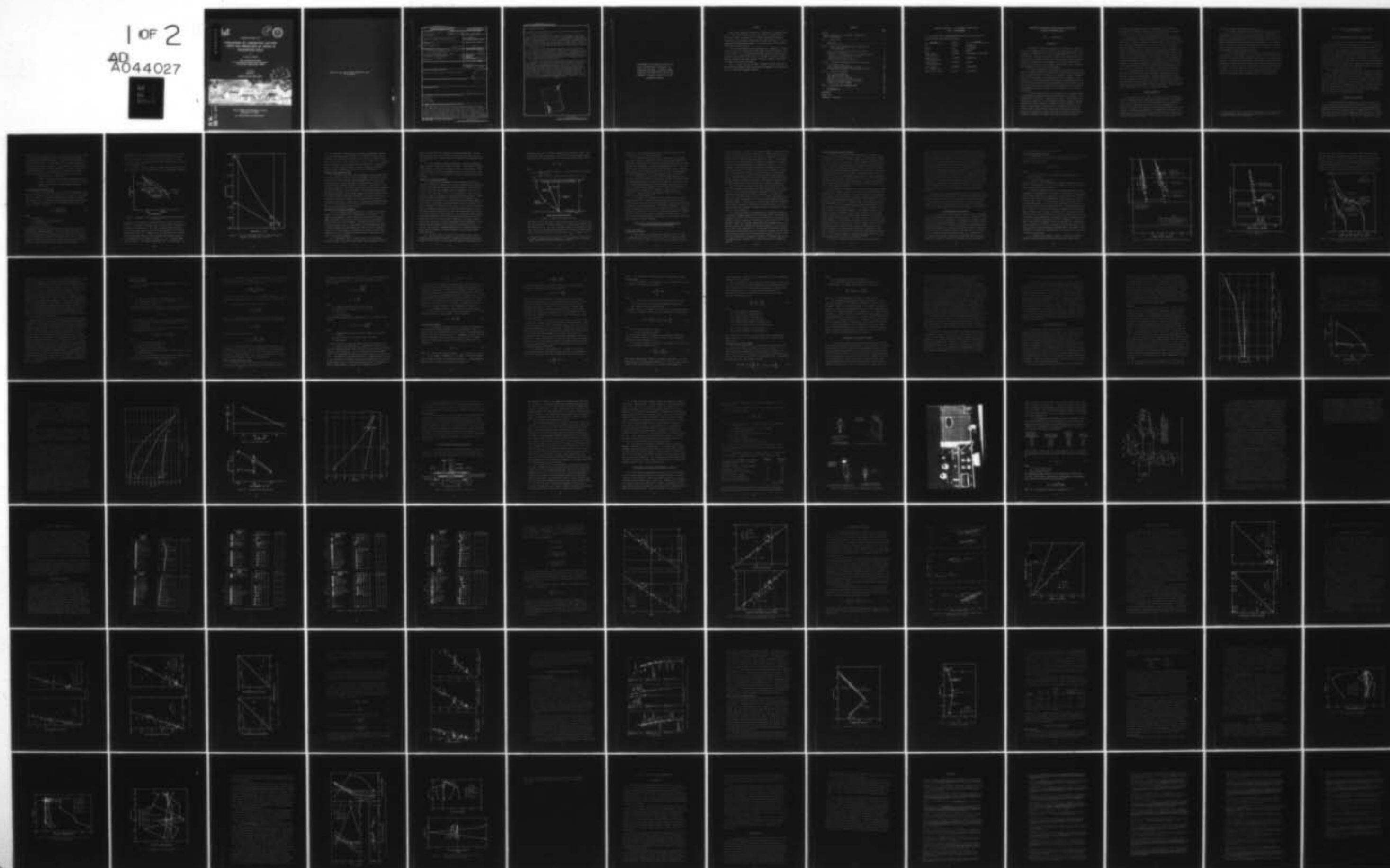

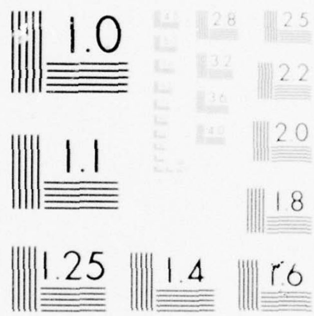

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EVALUATION OF LABORATORY SUCTION TESTS FOR PREDICTION OF HEAVE IN FOUNDATION SOILS

by

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Final Report
August 1977

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Commonly used methods for determining swell potential and predicting volume change and in situ heave are evaluated from the standpoint of simplicity, economy, reliability of test data, and simulation of field conditions. Swell potential is often defined as the percent swell from in situ water content to saturation under a surcharge pressure of 1 psi, whereas in situ heave is often defined as the percent swell from changes in the initial to assumed equilibrium moisture and confining conditions in the field. Increasing degrees (Continued)

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of expansion or swell potentials usually correlate with increasing liquid limits and plasticity indices.

Methods for determining swell pressure and predicting in situ volume changes or heave often use results of swell tests performed on undisturbed samples in the 1D consolidation apparatus. A new method for determining swell potential and predicting in situ volume changes based on soil suction relationships is described herein. The suction method is simple, takes little time, requires inexpensive equipment, and may simulate important field conditions including effects of lateral pressure and mechanics of the heaving process more closely than swell tests.

The equation for determining swell pressure by the suction method is consistent with the results of previous research; the swell pressure is primarily a function of the dry density for a given soil. Comparisons of results of tests performed on undisturbed samples indicate that swell pressures computed by the suction method and denoted herein as suction swell pressures are similar to swell pressures measured from swell tests. The amount of swell pressure depends on the type of swell test.

The equation for determining volume changes and heave in soils by the suction method is consistent with the consolidation equation for calculation of settlements. Swell indices computed by the suction method are usually larger than swell indices measured from most swell tests. Predictions of volume changes and heave computed by the suction method may consequently be greater than those computed by other methods using results of most swell tests assuming identical equilibrium mositure and confining conditions. The suction index may also be used as an estimate of the relative degree of expansion of soils. For special cases in which the degree of saturation is one, the suction index approaches the magnitude of the compression index and can be used as an estimate of the compression index.

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PREFACE

This report completes the work unit "Properties of Expansive Clay Soils." The work unit was started in 1967 under the sponsorship of the Office, Chief of Engineers, U. S. Army, Directorate of Military Construction. The initial studies were performed under the U. S. Army Operations and Maintenance program. The studies were continued under RDT&E Work Unit AT40 A3 007.

The work reported herein was performed by Dr. Lawrence D. Johnson, Research Group, Soil Mechanics Division (SMD), Soils and Pavements Laboratory (S&PL), U. S. Army Engineer Waterways Experiment Station (WES). The report was reviewed by Mr. Walter C. Sherman, Jr., and Dr. Donald R. Snethen, Research Group, SMD, and Mr. Clifford L. McAnear, Chief, SMD. Mr. James P. Sale was Chief, S&PL.

COL G. H. Hilt, CE, and COL John L. Cannon, CE, were Directors of WES during the conduct of this study and the preparation of this report. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)

UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
mils	0.0254	millimetres
inches	25.4	millimetres
feet	0.3048	metres
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square inch	6.894757	kilopascals
pounds (force) per square foot	47.88026	pascals
tons (force) per square foot	95.76052	kilopascals
atmospheres (normal)	101.325	kilopascals

EVALUATION OF LABORATORY SUCTION TESTS FOR PREDICTION OF HEAVE IN FOUNDATION SOILS

PART I: INTRODUCTION

Background

1. Damages to structures from swell and shrinkage of foundation soils due to changing moisture conditions are common problems that occur frequently in many parts of the world, including extensive areas of the United States. Although the extent of damages from swelling soils has not been well recognized in many local areas, property losses have been estimated to exceed $2 billion annually in the United States alone.[1] Differential rather than total movements of foundation soils are generally responsible for major structural damage.

2. Overlying structures often induce heave in swelling soils because the natural transpiration of moisture by vegetation and evaporation from the ground surface is inhibited. The amount of heave or shrinkage depends on the thickness and characteristics of the foundation soils, the extent to which preconstruction field moisture conditions may be disturbed by the structure and the new environment, actual moisture conditions during the life of the structure, and changes in the distribution of overburden and lateral pressures in the foundation soils because of construction.

3. Reliable predictions of in situ heave can be extremely helpful in developing more effective and economical designs of structures to be constructed on foundations in expansive clay soils. The choice of stabilization procedures or soil treatments may also be guided by the magnitude of the heave predictions. However, reliable prediction of the amount of heave beneath various parts of a structure after a certain period of time is made difficult by the effect on heave of such soil variables as composition, permeability, and stratification and such environmental variables as climate, availability of water following

construction, surcharge pressure, and temperature. One of the more important field conditions, and one of the more difficult to determine, for example, is the final or equilibrium moisture profile.

4. Numerous procedures and laboratory tests have been suggested for determining swell potential and predicting in situ heave of foundation soils.[2] The swell potential is usually defined as the percent swell for certain specified conditions such as from natural water content to saturation under a constant surcharge pressure of 1 psi. Practical methods for determining swell potential must be simple, economical, and useful for locating soils that may cause a heaving problem and for indicating soils that may need more thorough testing, but swell potential is usually not a reliable indicator of in situ heave. Swell potential has also been applied toward determining soil treatments to reduce foundation heave.[2] Laboratory swell tests simulating a variety of field conditions are frequently performed to provide better understanding of the heave behavior of foundation soils and for predicting in situ heave.

5. No procedure either for determining swell potential or for predicting in situ heave has been shown fully satisfactory or preferable for all practical applications. Adequate studies have not been made that compare the usefulness of different procedures for determining swell potential and predicting in situ heave. Some procedures, for example, require special laboratory tests and equipment. Other procedures may be applicable only in certain locations and climates.

Purpose and Scope

6. The purpose of this report is to present (a) an evaluation of various laboratory tests for measuring swelling properties of foundation soils and (b) comparisons of some well known methods for determining swell potential and predicting in situ heave of foundation soils. Methods are indicated that may be more practical and reliable than other methods. A new suction technique for determining swell potential and predicting in situ heave is investigated that may be more practical and reliable than existing methods. The suction method uses results from

relatively economical and simple laboratory tests.

7. Procedures studied herein for determining swell potential and predicting in situ heave use data obtained from soil classification tests; swell tests performed in the one-dimensional (1D)* consolidation apparatus and hereinafter referred to as consolidation swell (CS) tests; pressure membrane cells; and thermocouple psychrometers. Soil classification tests include specific gravity, natural water content, Atterberg limits, and percent clay content less than 2 μ. A variety of CS tests were performed to permit comparison studies.

8. The pressure membrane cell is capable of measuring matrix soil suction, which can be related to the water content and void ratio of undisturbed soils under vertical pressures simulating in situ overburden pressures. Thermocouple psychrometers measure relative humidity of soil specimens from which total soil suction-water content relationships may be evaluated. Total suction is defined as the sum of matrix and osmotic components of suction (Table 1).[3] Osmotic suction arises from the concentration of soluble salts in the pore water, while matrix suction arises from capillary forces and water fixation by polar adsorption.

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* For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix A).

PART II: METHODS FOR DETERMINING SWELL POTENTIAL AND PREDICTING HEAVE

Basis of Reliability and Practical Application

9. In situ heave is a function of field conditions such as soil moisture, weather, and confining pressures at the start of construction and changes in these conditions at subsequent times during the life of the structure. Exact simulation of these conditions in the laboratory is not possible. Various approximating methods are consequently followed for determining swell potential and predicting in situ heave. The reliability of methods for determining swell potential and predicting in situ heave may be related to the degree of correlation with measurements of the field heave of foundation soils beneath structures and pavements. The extent of practical application of reasonably reliable methods may be related to economy and degree of simplicity.

10. Standards are defined below for rating methods for determining swell potential and predicting heave based on the degree of simplicity, economy, and simulation of field conditions. Methods for determining swell potential and predicting heave are thereafter compared with reference to these standards. A new method based on soil suction behavior is proposed for determining swell potential and predicting heave that may simulate field conditions as well as or better than other methods. The new method is also relatively simple and as economical as many other methods for determining swell potential.

Standards for Comparison

11. The most important comparison of a system for rating different methods of predicting heave of foundation soils is the degree of conformity with heave observed in the field. Actual measurements of heave made in the field from swelling of foundation soils are scarce. Three field test sections have consequently been constructed to obtain field measurements of heave for comparison with predictions of heave

based on results of laboratory tests. (The results of this field study on test sections constructed at the U. S. Army Engineer Waterways Experiment Station (WES) installation at Clinton, Miss.; Lackland Air Force Base (LAFB), Tex.; and Fort Carson, Colo., will be published in a future report as part of the work unit "Foundations on Swelling Soils," sponsored by the Office, Chief of Engineers, U. S. Army, Directorate of Military Construction.) The standards for comparison of a rating system presented herein, in view of limited field data, include:

- a. Amount and type of data needed.
- b. Degree of simplicity and economy in obtaining the data.
- c. Degree of reliability of data considering test equipment.
- d. Degree of simulation of the environment and field conditions.

Data needed for heave predictions

12. Heave in the field may occur as a result of changing moisture and pressure conditions due to construction of a structure on the expansive soil foundation. Foundation soils may swell under the weight of the overlying soils and structure when adequate moisture becomes available. Data needed to predict in situ heave include the initial and final void ratios of the soil

$$\Delta H = H\left(\frac{e_f - e_o}{1 + e_o}\right) \tag{1}$$

where

ΔH = heave, ft

H = thickness of the clay stratum, ft

e_f = final void ratio

e_o = initial void ratio

The initial void ratio is easily obtained from laboratory tests on undisturbed soil samples taken prior to construction. The final void ratio is often obtained from estimates of the final effective pressure and the void ratio-log pressure relationships of the soil. The final effective pressure is based on the stress distribution and assumed pore pressures. Predictions of the rate of heave may also require

permeability and soil suction data or laboratory data curves of dimensional change versus time caused by wetting or changes in effective pressure. (Evaluation of procedures for predicting rates of heave will be the subject of a future report as part of the work unit "Foundations on Swelling Soils.")

13. Figure 1 shows a schematic of swell data obtained from a constant volume swell (CVS) test performed in a 1D consolidation apparatus.

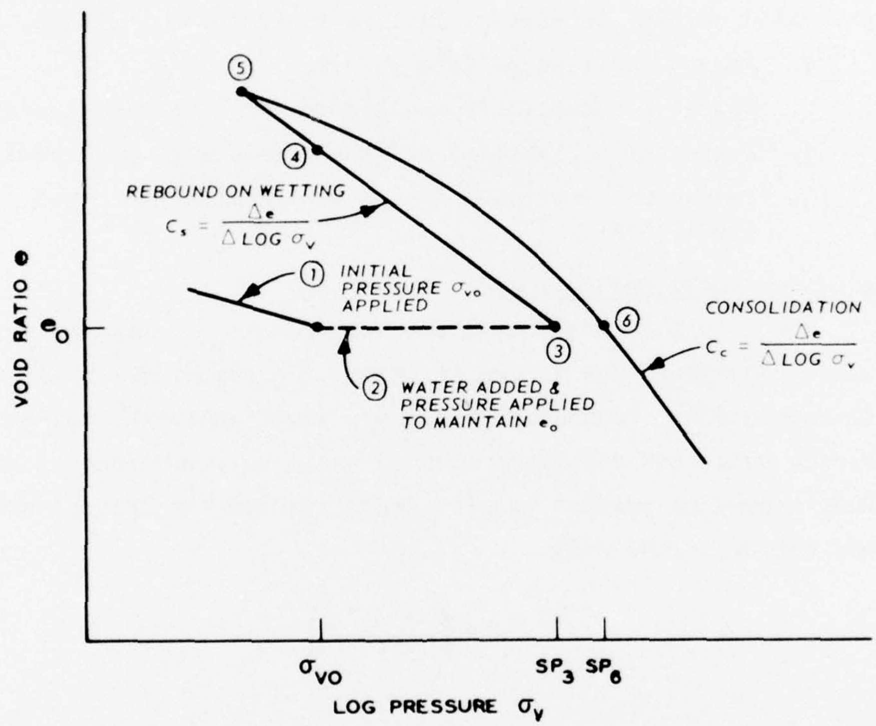

Figure 1. Example of the void ratio-log pressure relationship from a CVS test

An example of swell data for a specimen from Fort Carson is given in Figure 2. The numbers in Figures 1 and 2 refer to the time sequence of each phase of the CVS test. The slope of the appropriate portion of the void ratio-log pressure curve for predicting heave is often given by the swell index C_s. Computation of settlements due to consolidation from pressures greater than the maximum past pressure may use a slope indicated by the compression index C_c. C_s is almost always much less than C_c for virgin compression, but C_s can approach C_c

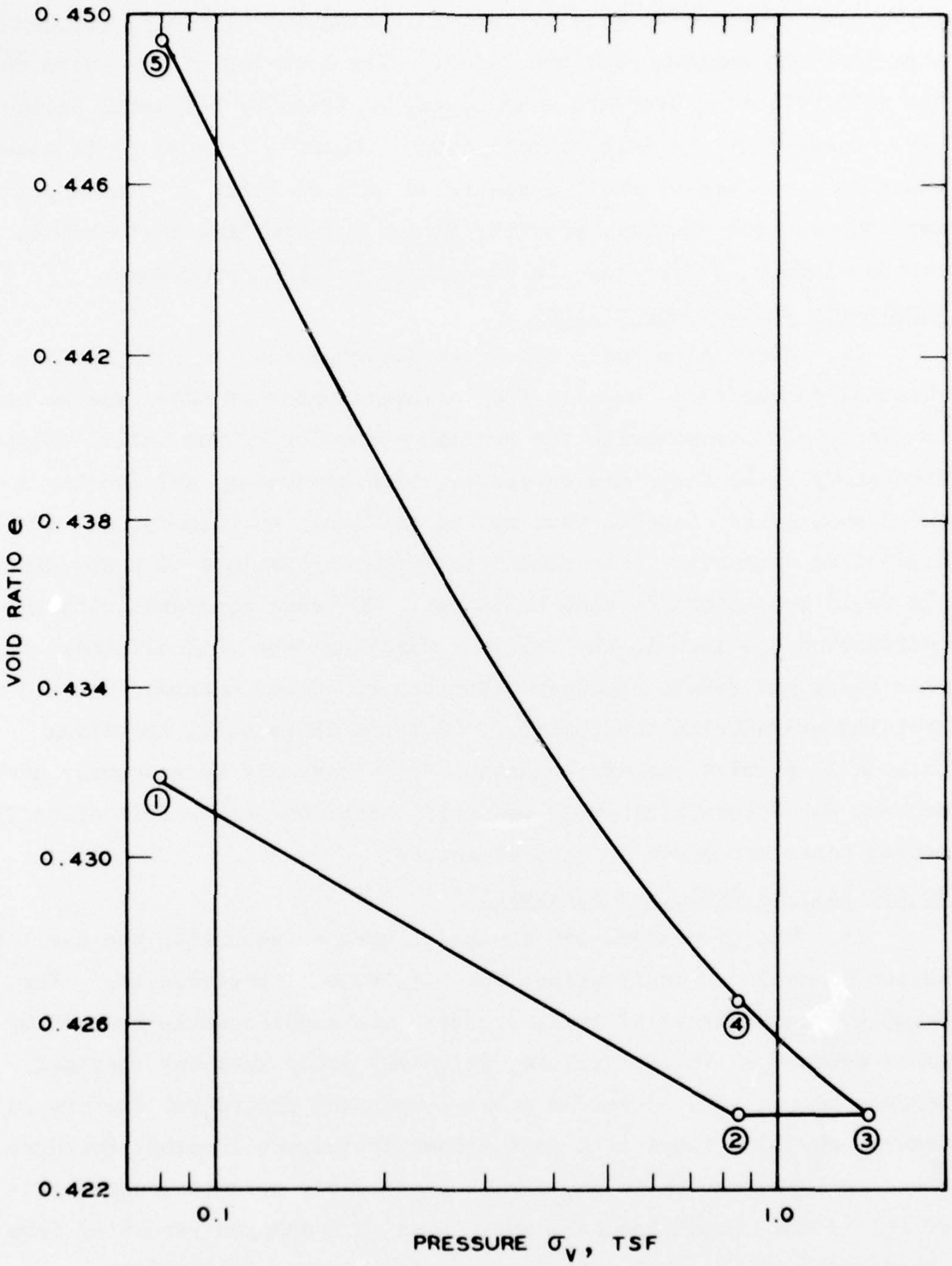

Figure 2. CVS test of specimen from Fort Carson, Boring P-4, Sample No. 9, depth 14.6 to 15.7 ft

for soils subjected to many cycles of loading and unloading (pressures less than the maximum past pressure).[4] The location of the curve on the void ratio-log pressure diagram may be fixed by the swell pressure (SP) measured at the initial void ratio. Point 6 (Figure 1) is sometimes taken as a measure of swell pressure as well as Point 3. These points may not coincide exactly, possibly because of sampling disturbance, stress history, differences in structure, and test procedure.

Simplicity and economy of test

14. Foundation soils are often heterogeneous in both lateral and vertical dimensions. Results from a large number of tests may be needed to adequately characterize the swelling behavior of the soils. Most laboratory swell tests are expensive, time-consuming, and usually require undisturbed samples that may be difficult to trim for the 1D consolidation apparatus. The extent to which the various CS tests simulate the field conditions is also not known. CS tests conducted with the undisturbed specimen in the original sampling tube will eliminate trimming costs and reduce specimen disturbance. Other methods that aid in avoiding or reducing the number of CS tests while still providing reasonable predictions of foundation soil heave may be extremely useful. Methods for determining swell potential based on simple soil classification tests are steps in this direction.

Reliability of data from equipment

15. The procedures and equipment used to determine the swell behavior of soils strongly affect the magnitude of the results.[5] For example, measurements of swell pressure are significantly reduced by small expansions in soil volume, which may occur when the specimen becomes seated in a consolidometer. Important procedural factors in testing swelling clays in a consolidometer include loading procedure (loading increment ratio and duration of load), friction, compressibility of the consolidometer, and dimensional changes resulting from compression of the soil against the porous discs, filter paper, and sides of the specimen chamber.

16. Considerable time is usually needed for consolidometer testing to reach equilibrium on wetting at various surcharge pressures;

hence, thin specimens are desirable to reduce testing time. Friction may cause large errors in observed loading at small loading pressures. Consolidometers with a high stiffness are preferred when measuring swell pressure.

17. Test equipment should be calibrated to correct the measured swell, especially with small changes in volume. Filter paper is highly compressible and should not be used in CS tests. Porous stones should be ground smooth to reduce displacement of the soil into grooves on the surfaces of the stones. The temperature should be noted and maintained constant.

Simulation of field conditions

18. Significant environmental or field conditions that may be simulated include soil composition, structure, stratification, pressure, and the initial and equilibrium soil moisture profile. Further details of field conditions that influence heave of foundation soils are described in the literature.[2,6-8] The soil composition, structure, and initial moisture condition may be simulated by testing specimens trimmed from undisturbed soil samples or, in the case of fill materials, by testing specimens compacted to a condition identical with that expected in the field. Sufficient undisturbed samples should be tested to adequately represent the behavior of the in situ soils. Stratification and vertical surcharge pressures may be simulated by performing CS tests at the field surcharge pressure on soil specimens taken from each distinctive stratum observed in the boring log. The undisturbed samples should be obtained from the field by reliable methods and properly preserved and protected.[2,6,9] The total heave predicted at the ground surface or base of the foundation is found by summing the heaves (Equation 1) contributed by each stratum for the final effective pressure at each stratum. The effects of lateral pressures are not simulated during CS tests, and the influence of lateral pressures is not evaluated when predicting heave.

19. The final effective pressure is calculated in part from an assumed equilibrium moisture profile. Methods of estimating the equilibrium moisture profile or final pore water pressure distribution with

depth that may occur in the field commonly use a saturated profile (zero negative pore water pressure) or a profile of decreasing negative pore water head with increasing depth from the ground surface (Figure 3)[6,10]

$$\tau_{mh} = \tau_{mh_A} + h \tag{2}$$

where

τ_{mh} = negative head at distance h above the depth of the active zone h_A , ft

τ_{mh_A} = negative head at the depth of the active zone h_A , ft

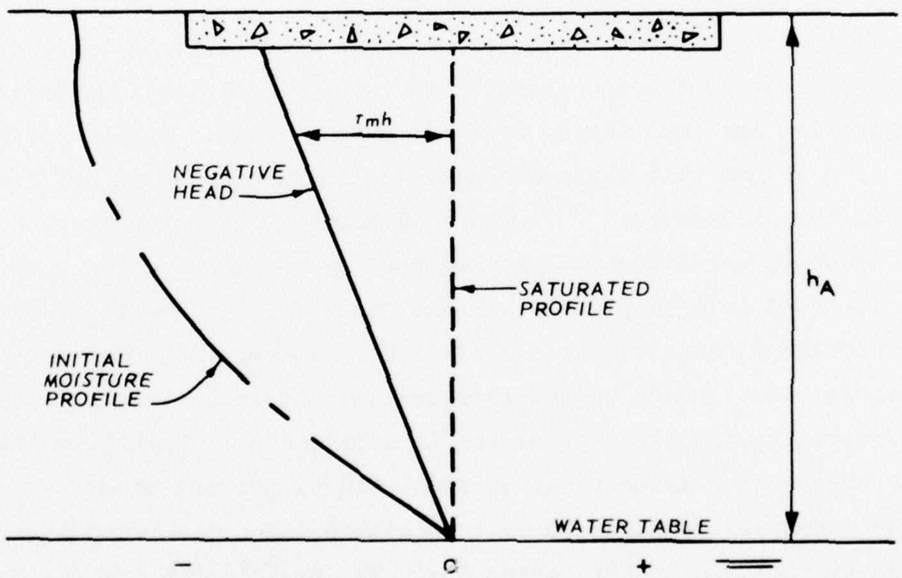

Figure 3. Examples of moisture profiles

The matrix suction head τ_{mh} is positive and equal to the negative pore water pressure head. Moisture conditions are stable below the depth of the active zone h_A . The h_A , although difficult to determine for deep water tables, is often assumed identical with the depth of shallow water tables (less than 20 ft* in clay soils) where τ_{mh_A} is equal to zero

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 4.

(Figure 3). A rising water table can also lead to heave due to the resulting decrease in effective pressure.

20. A saturated profile with zero pore pressure is not technically stable because of gravity forces, but such an approximation is reasonable in local areas near houses or buildings subject to excessive watering, poor drainage, or leaking underground water lines. The soils beneath slabs of buildings placed on the Permian red clay of Oklahoma were found completely saturated after about 20 years although means of wetting from leaking water lines and drainpipes were not found.[11,12]

21. Equilibrium soil moisture beneath highway pavements may be much dryer than beneath houses or buildings since water and sewer pipes and other man-made sources of water will probably not be present. Soil suction heads beneath pavements in Saskatchewan, Canada, were on the order of or greater (more negative) than the heads given by Equation 2 beneath centers of pavements but tended to be small near the shoulders.[13] Further details of estimating equilibrium moisture profiles are given elsewhere.[6,10]

22. The degree of saturation of soil at equilibrium is often assumed equal to one. Then, the effective pressure may be calculated by the principle of effective stress from known or estimated equilibrium surcharge and pore water pressures. Additional information is needed to relate the void ratio to soil suction and water content if the degree of saturation is less than one for soils at equilibrium moisture. Examples of field conditions under which the degree of saturation may be less than one at equilibrium include soil profiles with deep water tables and soils located in semiarid climates.

Comparison of Existing Methods for Determining Swell Potential and Predicting In Situ Heave

Methods for determining swell potential

23. Most methods of determining the swell potential or degree of expansion were described in an earlier report.[2] Several of these methods[7,14-22] given in Table 2 were selected for comparison studies.

The swell potential is defined in various ways (Table 2), usually according to the pressure and moisture conditions. A common definition is the percent swell (PS) from natural water content to saturation (zero pore pressure) under a constant surcharge pressure of 1 psi. The surcharge pressure of 1 psi may represent the loading on foundation soils exerted by concrete slabs and many small structures such as houses or 1-story buildings. The additional pressure exerted by overlying soils is not considered; therefore, the swell potential applies primarily to surface soils. A more general definition of swell potential is the percent swell of an undisturbed specimen from natural water content under the initial confining pressure to a state of saturation under the confining pressure expected following construction. The degree of expansion is a qualitative expression of the relative swell potential between soils.

24. The degree of expansion may often be correlated with soil classification data such as Atterberg limits (liquid limit (LL), plastic limit (PL), and plasticity index (PI)), shrinkage limit (SL), linear shrinkage, clay content (C), initial void ratio e_o, initial dry density γ_d, and initial water content w_o. Table 2 essentially indicates that clays with greater LL's or PI's are more probable sources of swelling problems. Any fat (CH) clay and even many lean (CL) foundation clays, however, can lead to damages in overlying structures from differential heave or shrinkage in foundation soils with the proper combination of field conditions.

25. Other procedures for determining swell potential include the PVC (potential volume change) meter,[20] expansion index,[7] and dielectric dispersion test.[21,22] The swell pressure indicated by the PVC meter will probably be underestimated due to expansion of the proving ring.[23] The expansion index test, although tedious and time-consuming, may be a potentially useful basic measure of the swell potential of soils independent of most field conditions.[7,21] The dielectric dispersion test is a promising and potentially simple and economical technique of determining a swell potential related to the water-holding capacity of soils. The dielectric dispersion is primarily a function of the amount and type of clay mineral, and it appears to be linearly related to the expansion index.[21]

Methods for predicting in situ heave

26. Many of the distinctly different types of methods for predicting in situ heave are described in Table 3. These methods attempt to account for the effect of various field conditions on heave, whereas the swell potential is a measure of heave independent of most field conditions. A swell potential defined as the percent swell from natural water content and confining pressure to saturation under the confining pressure following construction (paragraph 23) may provide a conservative or maximum prediction of in situ heave. Methods in Table 3 often use results of CS tests for calculating volume changes at each potentially troublesome clay stratum. Most methods assume that all of the volumetric swell will develop in the vertical direction. Fissures may reduce lateral pressures and limit swell in the vertical direction to about one third of the volumetric swell.[10]

27. Any method for predicting in situ heave must make some assumptions about the final pore pressures. The effects on pore pressures of such factors as climate, changes in water table elevation or piezometric pressure heads, and availability of water following construction of the structure are usually ignored during CS tests. Most methods for predicting heave assume either (a) a saturated final profile (zero pore pressures)[25,28,30,31] or (b) a final profile of decreasing negative head with increasing depth (Equation 2),[26,27,29] as shown in Figure 3. Reliable methods for predicting droughts or wet weather seasons, availability of water, and changes in water table elevation are not known. Reliable predictions of in situ heave are therefore not yet possible, except perhaps for limited cases. A common procedure is to assume some maximum probable pore pressure profile such as one of the above two cases and assume that the various causes of moisture changes will not lead to significantly greater pore pressures than the assumed profile. The structure should be constructed and maintained in a manner that will minimize changes in soil moisture.[6] Field data often indicate a steady change in soil moisture toward an equilibrium profile, except for seasonal influences near the edges of pavements.[8,37-39]

28. Results of improved simple oedometer (ISO) and CVS tests

appear overall to provide the most useful information on swelling properties for predicting in situ heave for a range of equilibrium moisture profiles and surcharge pressures. Predictions of heave based on results of double oedometer (DO) tests may be high because these tests allow water at zero pore water pressure to enter the soil specimen before significant surcharge pressures are applied, whereas moisture in the field enters the various soil layers under the restricting action of the field confining and negative pore water pressures.[40] In some cases, the DO test has significantly overestimated heave. The more recent ISO test eliminates some deficiencies of the DO test. Jennings et al.[27] advise that the ISO and DO tests were developed to avoid incomplete swell due to closure of fissures under the confining pressure and to more closely simulate field conditions. The lateral pressures in CS tests may also be very different from those existing in the field. Predictions of heave from ISO and some DO tests compared well with observed field heaves.[27]

29. Results of controlled suction tests[32] may be more applicable in heave calculations, particularly if the expected equilibrium moisture profile will not result in soils with a degree of saturation equal to one. Controlled suction tests performed with the pressure membrane apparatus can have the disadvantages of being difficult to perform, tedious, and time-consuming, and these tests require special equipment.

Predicting Heave from Suction

30. A new method of predicting heave based on soil suction-water content relationships is proposed that may reduce or eliminate the need for CS tests in 1D consolidometers. The equipment is extremely simple to use and is relatively inexpensive. The method can consider the composition, structure, stratification, surcharge and lateral pressures, and moisture conditions. Data from these suction tests are applicable to most moisture and loading conditions that may occur in the field. Also, the degree of saturation need not equal one. Observed soil suction-water content and volume-water content behavior patterns are applied to determine a suction swell pressure and suction index of

soils that may be used for predicting heave.

Soil suction and water content

31. Measurements of total soil suction with thermocouple psychrometers at atmospheric pressure indicated that matrix suction can be related to water content by[6]

$$\log \tau_m^o = \bar{A} - \bar{B}w \tag{3}$$

where

τ_m^o = matrix suction of soil without surcharge pressure, atm

$\bar{A},\bar{B}$ = constants

w = water content, percent

The reliability of these measurements is usually limited below 1 atm of suction. The parameters $\bar{A}$ and $\bar{B}$ are dependent on the clay mineralogy or composition and the structure.

32. Typical examples of the soil suction-water content curves determined on approximately 1-in. pieces of undisturbed Pierre shale and Upper Midway clayey shale illustrate the linear relationship (Figure 4). Figure 5 shows similar relationships for a sandy lean clay from Kelly Air Force Base (KAFB). An osmotic suction may be indicated in soils containing appreciable salts in the pore water when wetting these soils with distilled water (Upper Midway, Figure 4) or on drying (KAFB Sample No. 4, Figure 5). The soil from KAFB was dripping wet and had a wide range in natural water content. The osmotic suction can be subtracted from the total suction to yield matrix suction. Substitution of total suction may be more appropriate in Equation 3 when simulating field conditions if the water available to the soil is rainwater or distilled water. The effects of osmotic suction on heave may not be observed if the water external to the soil is identical with the pore water or if the pore water contains negligible dissolved salts. The nature of the influence of the type and concentrations of dissolved salts on heave is not yet well understood.

33. An appreciable change in slope $\bar{B}$ could not be detected between drying and wetting from the natural water content, which

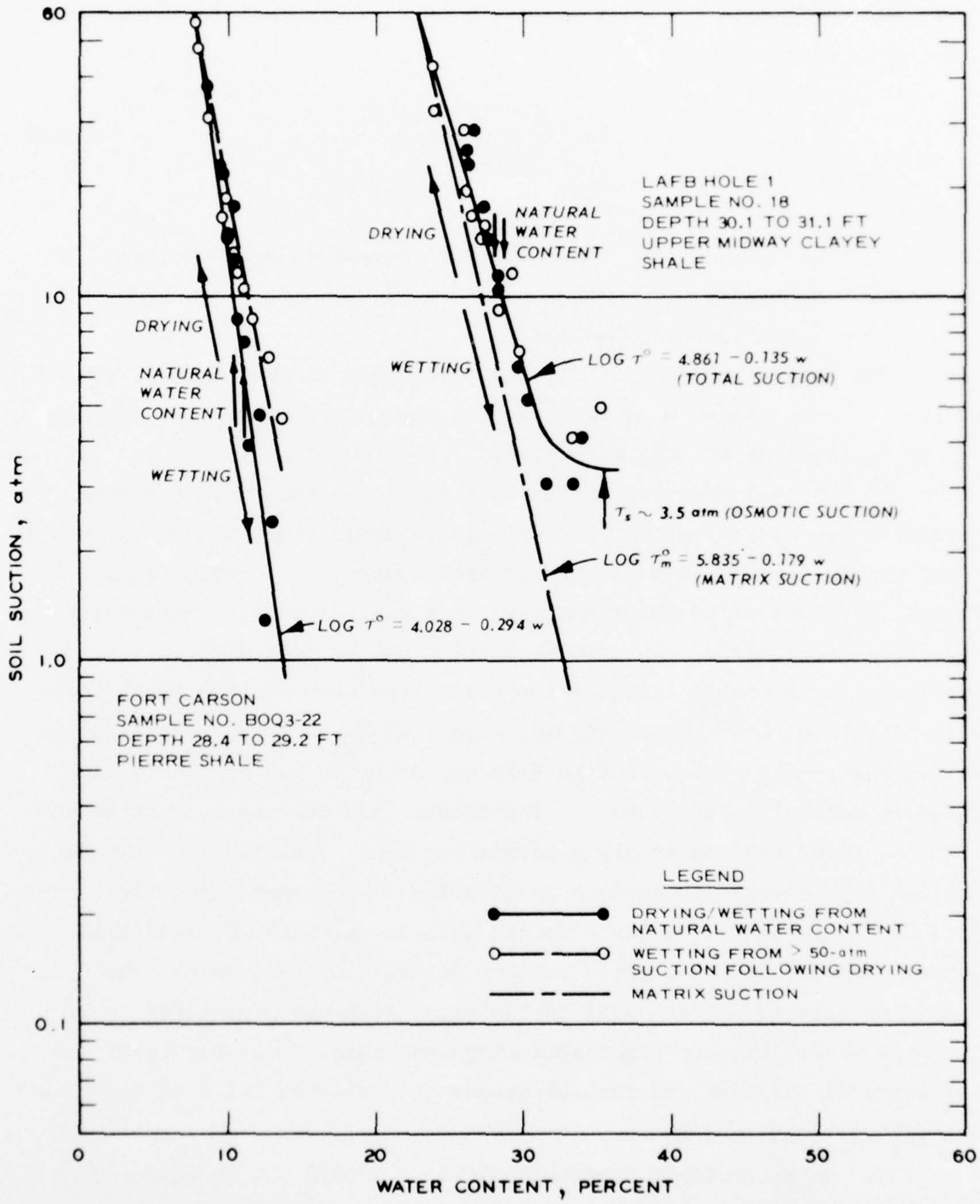

Figure 4. Soil suction-water content relationships of two shales

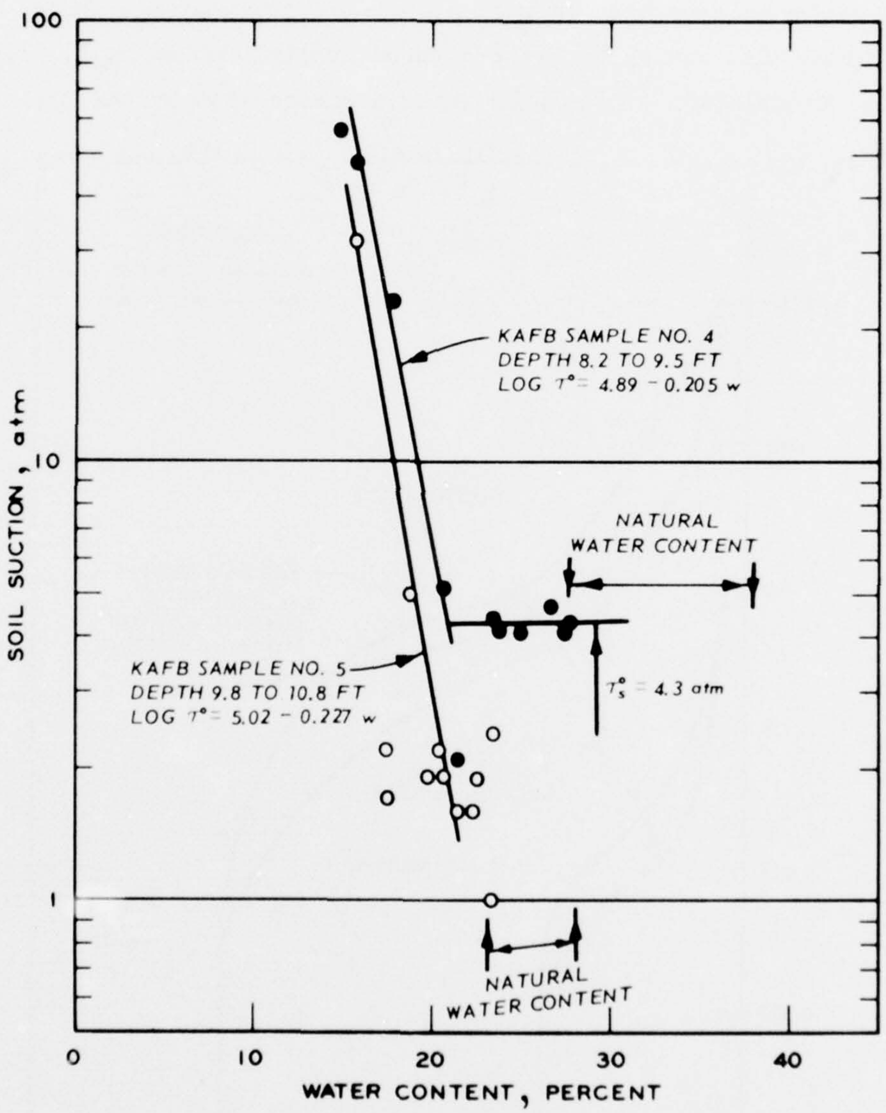

Figure 5. Soil suction-water content relationships of a sandy clay (CL) from KAFB

indicates that these shales do not have significant hysteresis between wetting and drying at atmospheric air pressure under zero confining pressure (Figure 4). In contrast, hysteresis between wetting and drying occurs in matrix suction-water content relationships measured with the pressure membrane apparatus (Figure 6).[41-43] The apparatus forces changes in matrix suction by air pressures applied to the soil. This technique, translation of the pore water pressure axis by air pressure,

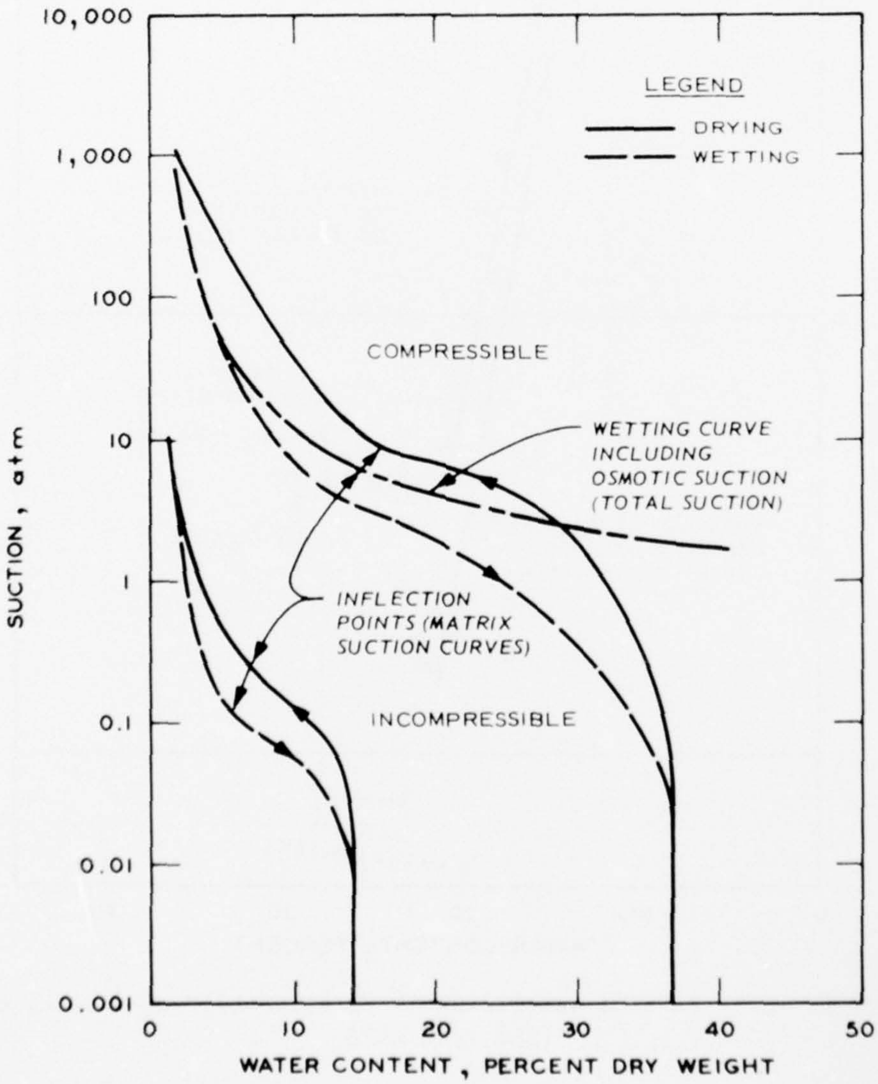

Figure 6. Examples of the suction-water content relationship evaluated by translating the pore pressure axis

is probably not the major cause of moisture changes in the field. Swell observed in specimens in contact with free water from reduction in vertical confining pressure during CS tests is also probably not representative of the cause of heave for many field cases. Changes in confining pressure due to construction often occur when free water is not available. For most in situ conditions, volume changes occur in swelling soils when water seeps into and out of the foundation soils, depending on the availability of water or amount of water evaporating from the ground surface, under constant confining pressure with any air voids at or near atmospheric air pressure. Suction-water content relationships determined from thermocouple psychrometers (Figures 4 and 5) are found by adding distilled water or by evaporating water from the samples. The latter procedure may be more representative of field conditions than other procedures when the effect of confining pressure on suction is applied to Equation 3.

34. The slope $\overline{B}$ of the relatively hard and unweathered Pierre shale appears reduced if the pieces of undisturbed soil are permitted to air-dry to a fairly high suction before wetting with distilled water (Figure 4). The reduction in slope compared to air-drying and wetting from natural water content is attributed to disturbance of the soil structure such as slaking due to wetting from an air-dried condition. A visual examination of air-dried soil and air-dried soil following wetting indicated disintegration of the structure and defoliation along laminated planes generally parallel to the horizontal plane. An appreciable difference in slope $\overline{B}$ was not observed in the fissured and weathered Upper Midway clayey shale. The difference in slope $\overline{B}$ may be useful when evaluating rebound in excavations due to disintegration of natural bonds in shales and subsequent swell. Schmertmann[44] observed increased swell in some soils when subjected to mechanical remolding, which he denoted as "swell sensitivity." The procedure of evaluating the $\overline{A}$, $\overline{B}$ parameters by wetting and air-drying from natural water content is used herein.

Effect of confining
pressure on suction

35. The effect of applying a confining pressure σ on τ_m^o is estimated by[42,45,46]

$$\tau_m = \tau_m^o - \alpha_\sigma \sigma = -u_w \tag{4}$$

where

τ_m = matrix suction under total applied pressure σ, tsf
α_σ = compressibility factor for applied pressure σ
u_w = pore water pressure, tsf

Confining pressures have no influence on osmotic suction. The effective stress $\bar{\sigma}$ when the degree of saturation is one is given by[47]

$$\bar{\sigma} = \sigma - u_w \tag{5}$$

When the degree of saturation is one such that for compressible soils α_σ is one, substitution of Equation 5 into Equation 4 shows that τ_m^o is the effective stress.

36. The matrix suction is reduced by both vertical and lateral total confining pressures such that

$$\sigma = \frac{\sigma_v + 2\sigma_h}{3} = \frac{(1 + 2K_o)\sigma_v}{3} \tag{6}$$

where

σ = total confining pressure, tsf
σ_v = total vertical pressure, tsf
σ_h = total horizontal pressure, tsf
K_o = coefficient of earth pressure at rest with respect to total pressure σ_h/σ_v

37. For soils below the water table and quasi-saturated soils, the in situ K_o may be determined from Equations 4-6

$$K_o = \frac{3\left(\tau_m^o + u_w\right) - \sigma_v}{2\sigma_v} \tag{7}$$

where u_w is determined from piezometric measurements and τ_m^o is determined from laboratory suction measurements. K_o for soils with degrees of saturation less than one may be estimated from

$$K_o = \frac{3\left(\tau_m^o - \tau_m\right) - \alpha_\sigma \sigma_v}{2\alpha_\sigma \sigma_v} \tag{8}$$

if in situ suction readings of τ_m can be obtained; otherwise, K_o may be roughly estimated from Equation 7 for similar soils below the water table.

38. From Equations 4 and 5 and the equation

$$\bar{\sigma} = \frac{\left(1 + 2\bar{K}_o\right) \bar{\sigma}_v}{3} \tag{9}$$

where $\bar{\sigma}_v$ is the effective vertical pressure, the coefficient of earth pressure at rest with respect to effective pressure $\bar{K}_o$ is given by

$$\bar{K}_o = \frac{3\tau_m^o - \bar{\sigma}_v}{2\bar{\sigma}_v} \tag{10}$$

Combining Equations 7 and 10 leads to

$$K_o = \bar{K}_o - \frac{\left(\bar{K}_o - 1\right) u_w}{\sigma_v} \tag{11}$$

The $\bar{K}_o$ of overconsolidated soils is commonly greater than one. K_o is therefore less than $\bar{K}_o$ below the water table for overconsolidated soils, but could be greater than $\bar{K}_o$ above the water table where u_w is negative. An upper bound to K_o for overconsolidated soils may be given by the coefficient of passive earth pressure.[47]

Effect of water content on volume

39. Changes in volume and water content are related by a compressibility factor derived by multiplying the unit weight of water γ_w

(in grams per cubic centimetre) by the slope of a curve relating the reciprocal of the dry density γ_d (in cubic centimetres per gram) to water content[45]

$$\alpha_s = 100\gamma_w \frac{\partial\left(\frac{1}{\gamma_d}\right)}{\partial w} \tag{12}$$

or

$$\alpha_s = 100 \frac{\partial V_T}{\partial w} \tag{13}$$

where

α_s = compressibility factor for volume changes

V_T = specific total volume;[41] i.e., $(1 + e)/G_s$

e = void ratio

G_s = specific gravity

Lytton and Watt[33] formulated an expression for α_s in terms of water content

$$\alpha_s = \alpha_o + (1 - \alpha_o)\left(\frac{w}{w_A}\right)^{Q-1} \tag{14}$$

where

α_o = compressibility factor for volume changes at zero water content

w_A = water content at air entry, percent dry weight

Q = an exponent

α_s will increase to one when the degree of saturation reaches one.

40. Lytton[41] has shown that α_σ (Equation 4), defined as the fraction of applied pressure that is effective in altering the pore water pressure, may be related to α_s, but that they are not necessarily equal. The compressibility factors are assumed equal herein and denoted as α to simplify the following derivations. Research beyond the scope of this report will be needed to more fully understand the relationship between these factors. If the compressibility factor is not known, α may be roughly taken as a constant and estimated from the PI[48]

$$
\begin{array}{ll}
PI < 5 & \alpha = 0 \\
5 \leq PI \leq 40 & \alpha = 0.0275PI - 0.125 \\
PI > 40 & \alpha = 1
\end{array}
\tag{15}
$$

A one-point check of Equation 4 can be made from the results of a triaxial compression test performed on Upper Midway clayey shale with thermocouple psychrometers included for measurement of total suction.[49] The suction was found to decrease approximately 80 percent of the increase in applied confining pressure, which corresponds to an α of 0.8. Atterberg limits tests indicated PI's ranging from 58 to 61 such that α should be one from Equation 15. The compressibility factor is overestimated about 20 percent for this example. Skempton's pore pressure parameter B appears analogous to the compressibility factor α_σ .

41. The change in specific total volume or void ratio with respect to water content may be approximated from Equation 13 as a linear relationship within a limited range of water content by

$$
\Delta V_T = \frac{\Delta e}{G_s} = \frac{\alpha \Delta w}{100} \tag{16}
$$

Suction swell pressure

42. If a pressure is applied to a soil specimen to maintain constant volume following submergence of the soil specimen in water of chemical composition identical with the pore water, the applied pressure can be defined as a swell pressure SP . Submergence in water with composition identical with the pore water eliminates effects of osmotic suction. The matrix suction under zero confining pressure will be (Equation 4)

$$
\tau_m^o = SP_s \tag{17}
$$

where SP_s is the suction swell pressure. τ_m is zero and α is one.

43. The degree of saturation S is equal to one at equilibrium after sorption of water at a constant void ratio e . The equilibrium water content in terms of void ratio will be given by

$$w = \frac{100Se}{G_s} = \frac{100e}{G_s} \qquad (18)$$

The suction swell pressure in terms of void ratio is given from substitution of Equations 17 and 18 into Equation 3

$$\log SP_s = \overline{A} - \frac{100\overline{B}e}{G_s} \qquad (19)$$

The suction swell pressure may be equivalent to the swell pressure determined from laboratory CS tests defined as above provided the swell tests are performed at constant volume until the specimen is fully saturated and the swell pressure is fully developed. Measured swell pressures and swells from laboratory swell tests will be misleading if the degree of saturation is originally low or the permeability is small such that the specimen does not reach full saturation during the course of even lengthy tests.

44. The suction swell pressure (Equation 19) is dependent on the composition, structure, void ratio, and specific gravity, which is essentially consistent with Chen's[50] observation that the swell pressure of a given soil depends only on the dry density. Equation 19 is also consistent with the expression for swell pressure derived from diffused double layer theory by Nishida, Nakagawa, and Koike.[51] According to the equation for swell pressure derived by Nishida, Nakagawa, and Koike, the slope $\overline{B}$ in Equation 19 should be a function of the valence and concentration of ions, the dielectric constant, the surface area of the particles, and the temperature. Effects of structure are not included in the diffused double-layer formulation.

45. The suction swell pressure SP_s in terms of the initial matrix suction without confining pressure τ_{mo}^{o} prior to submergence in water is derived by substitution of Equations 18 and 19 into Equation 3

$$\log \frac{\tau_{mo}^{o}}{SP_s^{S}} = \overline{A}(1 - S) \qquad (20)$$

where S is the degree of saturation prior to submergence in water.

Suction index

46. From Equation 3, a difference in total or matrix suction can be related to a change in water content

$$\log \frac{\tau^{o}_{mo}}{\tau^{o}_{mf}} = \bar{B}\Delta w \tag{21}$$

where

τ^{o}_{mo} = initial matrix suction without surcharge pressure, atm

τ^{o}_{mf} = final matrix suction without surcharge pressure, atm

Δw = change in water content, percent

The change in volume or change in void ratio for a corresponding change in suction is obtained by substitution of Equation 16 into Equation 21

$$\frac{\Delta V}{V} = \frac{e_f - e_o}{1 + e_o} = \frac{C_\tau}{1 + e_o} \log_{10} \frac{\tau^{o}_{mo}}{\tau^{o}_{mf}} \tag{22}$$

where

$\Delta V/V$ = fraction change in volume V

C_τ = suction index $(\alpha G_s/100\bar{B})$

The consolidation equation[4] is analogous to Equation 22 where the suction pressures converge to effective pressures when the degree of saturation is one (Equations 4 and 5).

47. The suction index is defined as the change in void ratio for a log cycle change in matrix suction

$$C_\tau = \frac{\alpha G_s}{100\bar{B}} = \frac{\partial e}{\partial \log \tau^{o}_{m}} \tag{23}$$

The suction index appears related to the swell index when α is less than one and to the compression index when α is one as shown later and could be used to predict swells and settlements from changes in

effective pressure. Results of tests performed in the 1D consolidation apparatus are not necessary.

48. The magnitude of the suction index may be used as a method of identifying the relative swelling capability of the undisturbed soil or used as a measure of the swell potential independent of field moisture and confining conditions. The swell sensitivity described by Schmertmann[44] can be given quantitatively in terms of the suction parameters from Equation 23 by

$$\frac{C_{sr}}{C_{su}} = \frac{C_{\tau r}}{C_{\tau u}} = \frac{\alpha_r \overline{B}_u}{\alpha_u \overline{B}_r} \tag{24}$$

where

C_{sr} = swell index of remolded soil
C_{su} = swell index of undisturbed soil
$C_{\tau r}$ = suction index of remolded soil
$C_{\tau u}$ = suction index of undisturbed soil
α_r = compressibility factor of remolded soil
α_u = compressibility factor of undisturbed soil
$\overline{B}_u$ = suction parameter of undisturbed soil
$\overline{B}_r$ = suction parameter of remolded soil

The suction index may also be useful in stabilization studies where smaller values of α or PI and larger values of $\overline{B}$ indicate more stable soils.

Determination of volume changes

49. Volume changes in a clay stratum may be estimated directly from Equation 22; however, Equation 22 is not exact because α is assumed constant or independent of pressure and water content. The compressibility factor may decrease with increasing confining pressure and may become negative, especially for collapsible soils. Substitution of Equations 3 and 4 into Equation 22 gives

$$\frac{\Delta H}{H} \approx \frac{\Delta V}{V} \approx \frac{C_\tau}{1 + e_o}\left[\overline{A} - \overline{B}w_o - \log\left(\tau_{mf} + \alpha\sigma_f\right)\right] \tag{25}$$

where

τ_{mf} = final matrix suction under pressure σ_f

σ_f = final applied pressure at the final void ratio e_f

For cases where S is always one, Equation 22 simplifies to

$$\frac{\Delta H}{H} = \frac{C_\tau}{1 + e_o} \log \frac{SP_s}{\tau_{mf} + \sigma_f} \qquad (26)$$

where C_τ is found from Equation 23 with α equal to one.

50. Some additional approximation may be necessary to compute heave from Equation 25 if the degree of saturation increases to one such that α may not be assumed constant. For example, α may be denoted as a function of water content (Equation 14) and integrated into Equation 25. Equation 26 might also be used with a C_τ determined from an average α based on results of laboratory and field studies. Since highly expansive soils tend to have α close to one,[42,48] substitution of one for α in the expression of C_τ (Equation 23) will not cause much error and will lead to conservative or safe predictions of heave. The α for less expansive soils could continue to be given by Equation 15 without causing much error. The final confining pressure σ_f should include any lateral pressures (Equation 6) in overconsolidated soils; otherwise, predicted heaves may be excessive.

Limitations of the Suction Method

51. The suction method has some apparent limitations in determining swell pressure and volume changes. At both high and low suctions, the suction-water content relationship will probably deviate from the empirical formulation (Equation 3). Deviations at high suction levels above 50 atm are probably of very little practical significance because most field pressures of interest are below 50 atm. Deviations that may occur at low suction levels of less than 1 atm, which cannot be accurately measured by the thermocouple psychrometer, may be extremely important and may lead to errors in predictions of volume changes for

shallow soils under light surcharge pressures. At low suctions, the osmotic suction will become dominant and will probably lead to a horizontal curve as illustrated in Figure 5 for KAFB Sample No. 4.

52. Equations 22 and 25 for predicting heave may not consider effects of soil strength. For example, particle bonding may have some undefined effect on swell pressures and swell. Suppose that a relatively dry specimen, containing soluble material cementing the soil particles together, is to be immersed in water while the specimen is loaded under a significant surcharge pressure. Water seeping into the specimen, with chemical composition different from the pore fluid, may dissolve the physical and chemical bonds causing the structure to collapse. The developing swell pressure may not be sufficient to react against the surcharge loading to support the weakened structure. Equation 19 may predict the swell pressure that will ultimately occur provided the structure does not change significantly (collapse) during the swell test; i.e., the surcharge pressure should be manipulated to maintain a constant void ratio and the external (free) water should be identical with the pore fluid. Equation 26, which includes the suction swell pressure, may account for volume changes in collapsible soils and may provide improved heave predictions when the initial degree of saturation is less than one.

53. Equation 19 may also be used to calculate a swell pressure in nonexpansive sandy and silty soils, but the meaning of such computations is not yet clear. Substitution of the surcharge pressure σ_{vo} in place of SP_s (Equation 19) may indicate the void ratio needed to eliminate subsequent swell; however, soil strength is not considered and settlement could occur. Apparently, a complete understanding of the effect of suction on the behavior of volume changes in soils may also depend on understanding the effect of suction on soil strength.

PART III: LABORATORY EQUIPMENT AND TESTING PROCEDURES

54. Three series of tests were performed to evaluate laboratory test procedures for determining swell potential and predicting heave of foundation expansive soils. The first series involved performing variations of CS tests on undisturbed specimens. Secondly, pressure membrane tests were performed to determine matrix suction-water content-void ratio relationships of undisturbed soils under vertical pressures simulating the in situ overburden pressures. Thermocouple psychrometers were used during the final series of tests to determine total or matrix suction-water content relationships of pieces of undisturbed soil of about 1-in. dimensions. Surcharge pressures were not applied during suction measurements with thermocouple psychrometers. Standard classification tests such as visual description, natural water content, Atterberg limits, specific gravity, and grain size distribution analyses were also performed on trimmings of undisturbed specimens prepared for the three series of laboratory tests.

Consolidometer Swell Tests

55. The types of swell tests performed in the 1D consolidation apparatus included the constant volume swell (CVS), improved simple oedometer (ISO), swell overburden (SO), and swell pressure (SP) tests. The selected tests provide a reasonable simulation of the different types of tests commonly performed on undisturbed swelling soils (Table 3). Predictions of heave from these tests assume that the final degree of saturation is one and that the final pore pressure is zero under the estimated in situ surcharge pressure. Time required to perform a CS test on a single soil specimen varied, for example, from about 2 to 3 weeks and 3 to 5 weeks for SO and CVS tests, respectively, or more depending on the type of soil and number of loading increments and decrements. Technician time was needed to trim and set up the specimens in addition to time needed for monitoring.

56. The undisturbed specimens for swell tests were identically

trimmed (i.e., 4.25 in. in diameter by 1.15 in. high) and seated in a 1D consolidometer between air-dry porous stones with a small seating load (approximately 0.02 tsf). Filter paper was not used to eliminate error from compression of the paper. The inside of the reservoir was moistened and the specimen and consolidometer assembly were covered with impervious plastic to maintain constant moisture conditions. All applied pressures prior to adding free water were held not more than 30 min to minimize loss of moisture. Testing indicated that specimens held for lengthy times prior to adding water will decrease in void ratio due to drying. Swell tests were performed with distilled water added to both top and bottom porous ceramic stones.

CVS test

57. The CVS test is illustrated in Figure 1. It begins with a seating load on the specimen, which is applied for 30 min. The overburden pressure σ_{vo} is subsequently applied and held for 30 min. Free water is added to the reservoir and sufficient load applied in small increments to prevent swelling until the swell pressure is fully developed. The submerged specimen is unloaded in decrements to the seating load. Each decrement should be held until primary swell is complete. The dimensional change with time relationships shown in Figure 7 for the specimen from Fort Carson (Figure 2) illustrate the lengthy, uneconomical periods of time required for swelling of some soils. The time permitted to achieve rebound normally would have been about 1 week (10,000 min) or less because primary swell appeared to have been completed within this time; however, long-term testing has shown that swell continues after 280 days (400,000 min). Figure 2 illustrates the modified CVS test that involves only two unloading decrements: one at the soil overburden pressure σ_{vo} and the other at the seating load. An alternative method in an attempt to reduce sample disturbance is to load the specimen to the maximum past pressure, reduce the load to the calculated σ_{vo} , and then add water as before (maximum past pressure CVS test).

58. Heave may be predicted from Equation 1. The final void ratio is determined from the final effective pressure and the void

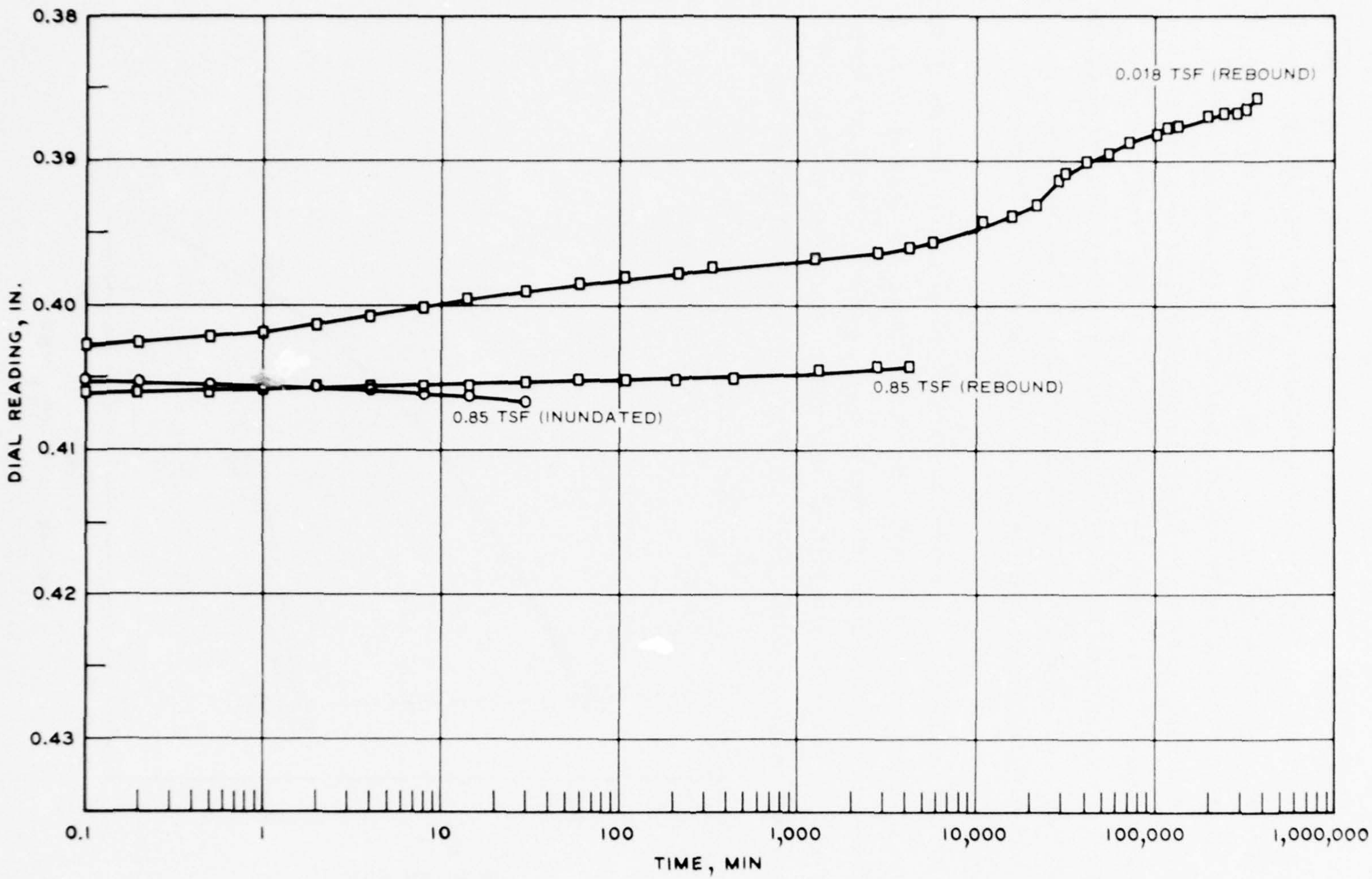

Figure 7. Time-expansion curves of specimen from Fort Carson, Boring P-4, Sample No. 9, depth 14.6 to 15.7 ft

ratio-log pressure relationship on the rebound curve (Figure 1). The final effective pressure is calculated by the principle of effective stress (Equation 5) from estimates of final surcharge and pore water pressure. The final pore water pressure may sometimes be estimated by assuming a negative head given by Equation 2. If the final effective pressure is greater than the swell pressure, the final void ratio may be estimated from an extension of a curve parallel to the consolidation curve (Point 6) from the swell pressure SP_3 at Point 3 (Figure 1).

ISO test

59. This ISO test suggested by Firth[27] is a simplification of the DO test.[26] The specimen is loaded for 30 min under the seating load, then loaded to the overburden pressure σ_{vo} in one increment and held for 30 min to determine the initial void ratio e_o (step 2, Figure 8). The pressure is subsequently reduced to the seating load in one decrement

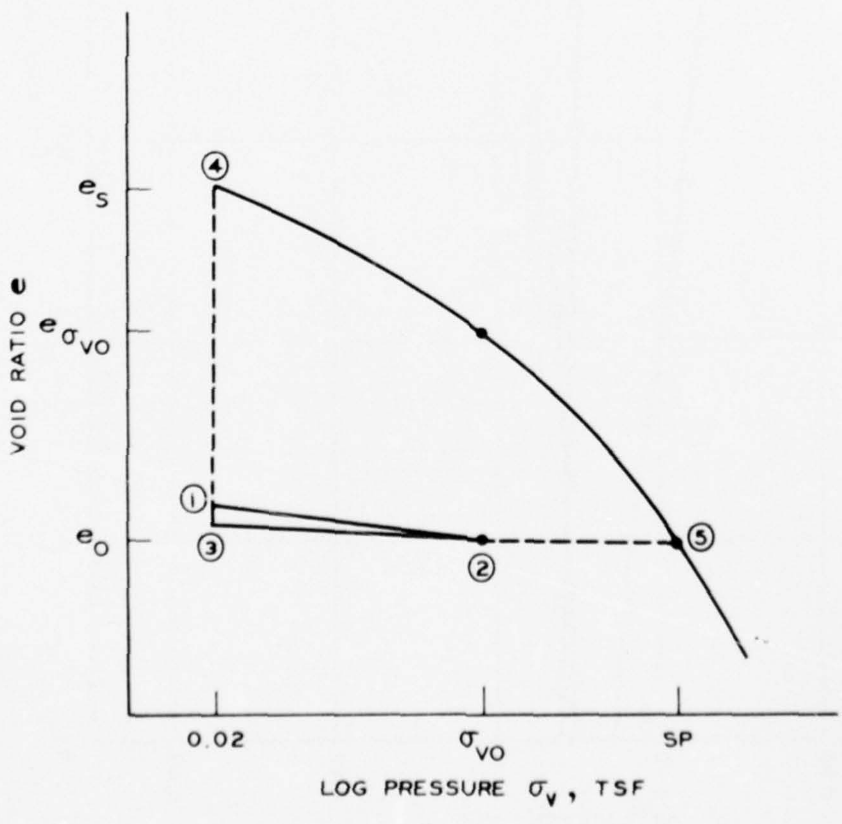

Figure 8. ISO test

and held for not more than 30 min (step 3). Free water is added and the swell under the seating load is observed until primary swell is complete (step 4). Increments of load are applied to achieve consolidation in excess of the initial void ratio. The applied pressure for consolidation to the initial void ratio e_o is sometimes taken as a measure of the swell pressure (step 5). Figure 9 illustrates ISO test results of a specimen from LAFB. Heave is predicted by a method similar to that for the CVS test, except that the consolidation curve is used instead of the rebound curve.

SO test

60. The SO test begins with the seating load, which is applied for 30 min. The overburden pressure σ_{vo} is applied in one increment and held for 30 min. Free water is added to the reservoir and the swell of the specimen is measured until primary swell is complete. Heave can only be predicted by Equation 1 for the effective pressure σ_{vo} applied during the test.

61. A variation of the SO test is the multiple SO test in which separate specimens are trimmed from adjacent positions in the soil sample and subjected to different surcharge pressures (Figure 10). The percent swell can be indicated as a function of surcharge pressure and heave may be predicted for a range of final effective pressures. A swell pressure can be evaluated as the pressure at which the percent swell is zero.

62. A modified SO test may also be performed in which increments of pressure are applied following primary swell (step 3, Figure 11) to achieve consolidation of the specimen back to e_o (step 4). The sample may also be rebounded to σ_{vo} (step 5) and then to the seating pressure (step 6). The void ratio at step 3 may be greater than the void ratio at step 5 due to hysteresis and the possibility that the alloted time for swell may be inadequate. The greater void ratio (step 3) should probably be used for a more conservative prediction of heave. Figure 12 illustrates modified SO test results of a specimen from Fort Sam Houston. The modified SO test permits a measurement of the swell pressure and a measure of the change in void ratio for a range of overburden pressures and effective pressures with a single soil specimen.

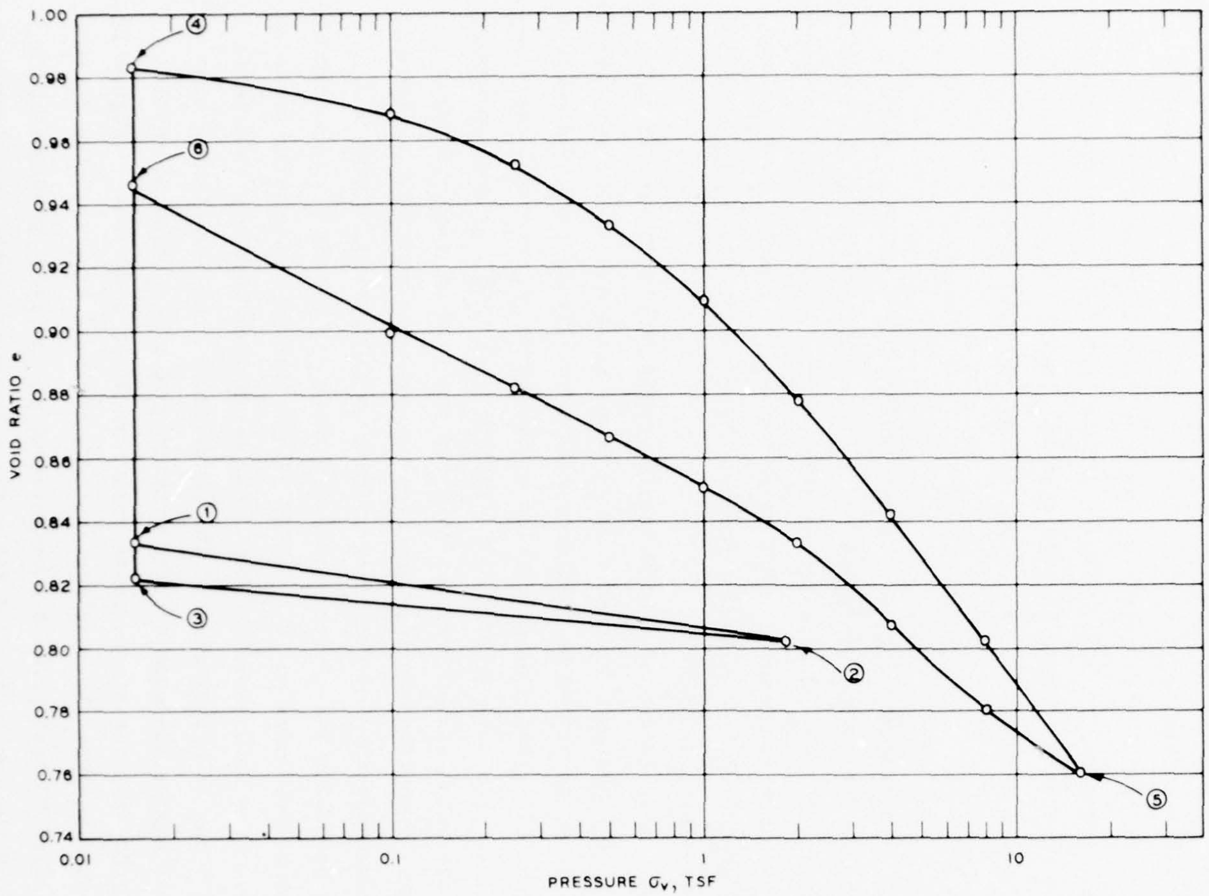

Figure 9. ISO test of specimen from LAFB, Boring 1, Sample No. 17, depth 29.0 to 30.0 ft

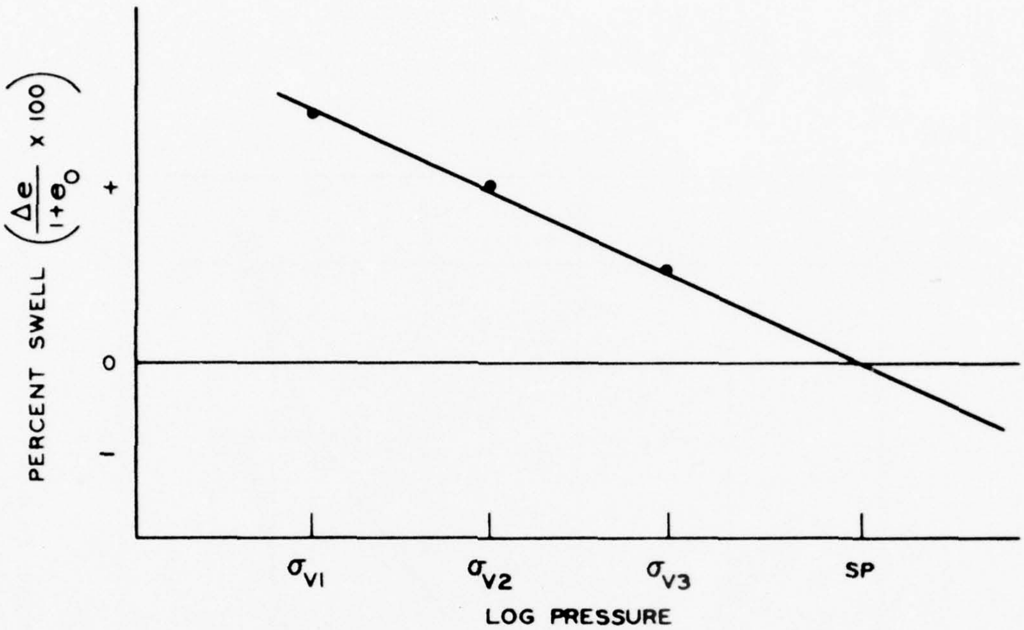

Figure 10. The multiple SO test

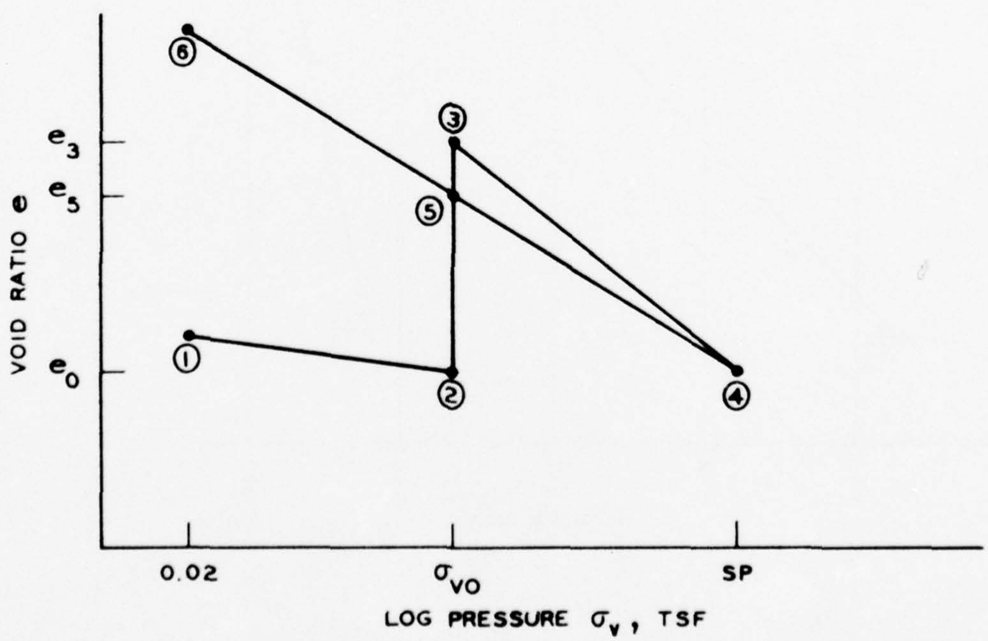

Figure 11. The modified SO rebound test

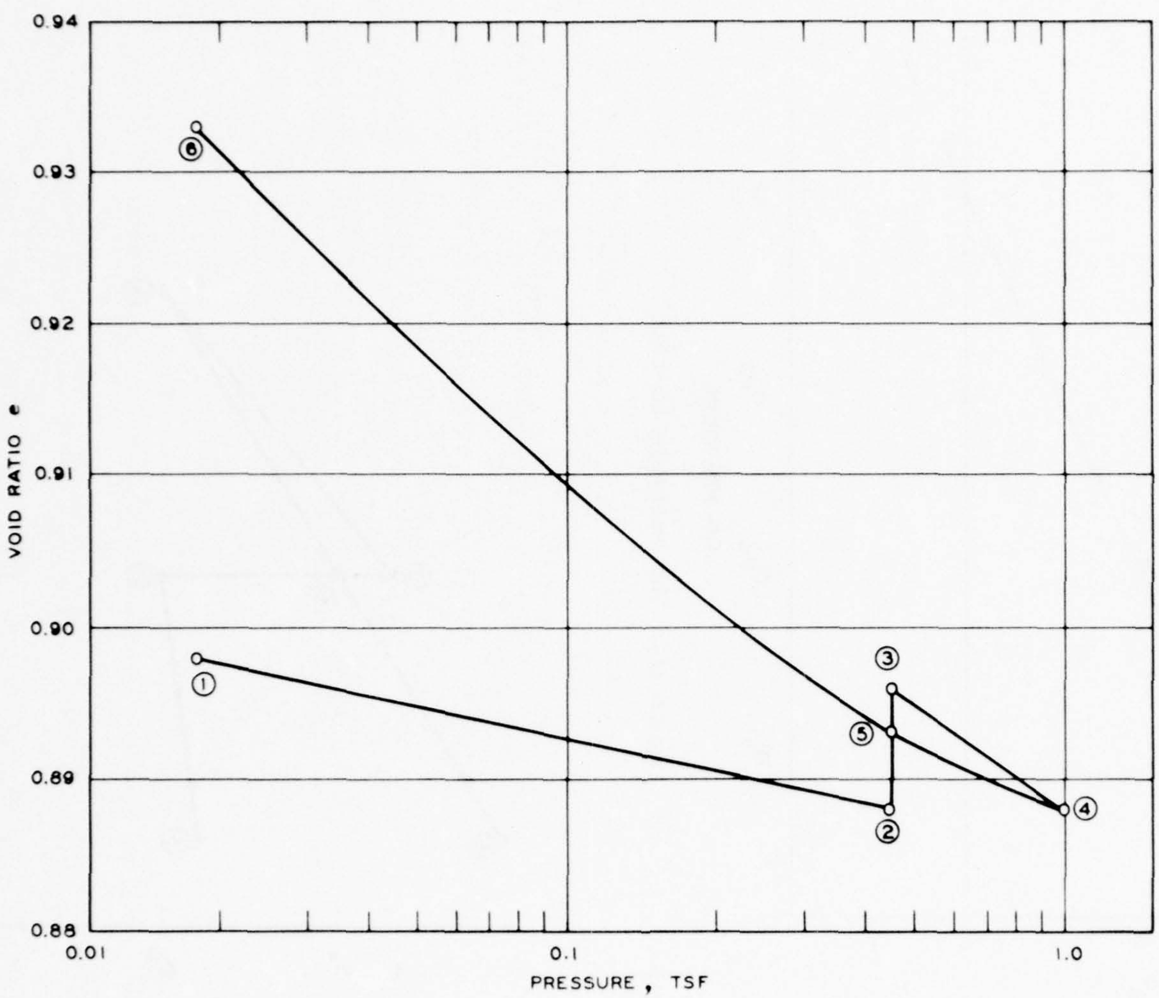

Figure 12. Modified SO test results of specimen from Fort Sam Houston, Boring 1, Sample No. 7, depth 7.2 to 7.9 ft

63. Heave predicted from the void ratio at step 3 (Figure 11) assumes a final pore pressure of zero. The rebound curve (points 4,5, and 6, Figures 11 and 12) can be used to predict heave for a range of effective pressures by the method used for analysis of CVS test results.

Swell pressure

64. The swell pressure (SP) is defined as the equilibrium pressure required for preventing volume expansion in the soil in contact with water. Further details describing various definitions of swell pressure are given in Reference 6. The SP determined herein is found by first applying the seating load to the specimen for 30 min. Distilled water is subsequently added and surcharge pressure is applied in small increments to maintain constant volume until the swell pressure is fully developed. Following determination of the swell pressure, the specimen may be unloaded in decrements to determine the rebound curve or the swell index.

Pressure Membrane Measurement of Suction

65. The pressure membrane is operated by applying a known air pressure to a soil specimen called the extraction air pressure P_e (Figure 13). The extraction air pressure elevates the negative pore

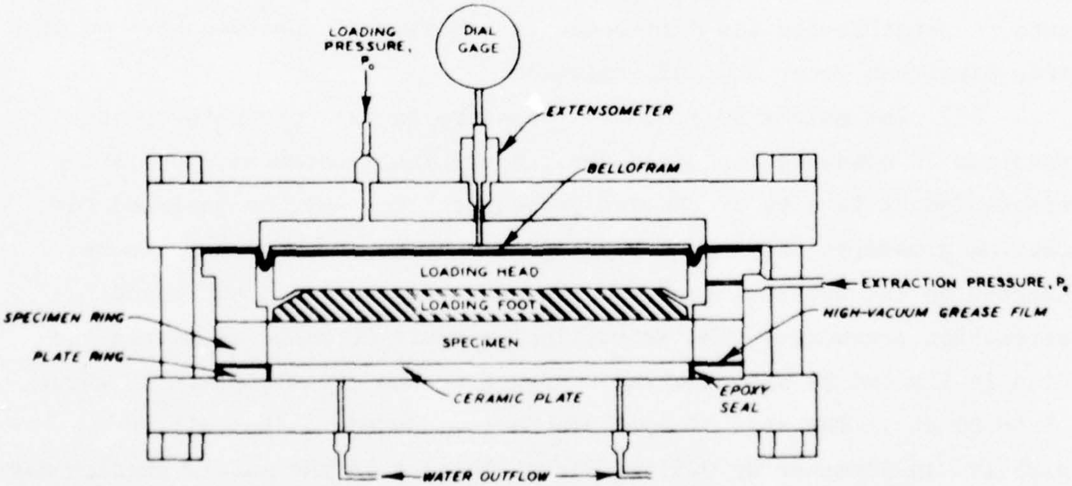

Figure 13. Negative pore water pressure cell

water pressure to induce a less negative or positive pore water pressure in the soil. If the pore water pressure becomes positive, water will flow or be extracted from the specimen through the membrane into the water outflow tubes. The water outflow tubes are connected to a graduated burette to permit measurement of the volume of water extracted from the specimen. The ceramic plate in Figure 13 serves as the membrane. The extraction pressure that results in no change in volume of water in the burette is the matrix soil suction.

66. The pressure membrane cell measures only matrix suction because the pore sizes in the ceramic plate are too large to prevent diffusion of the salts from the soil through the membrane into the free water. The salts will eventually become uniform in concentration throughout the pore and free water and the pressure from osmosis will not exist. The semipermeable membrane inherent within the soil due to the so-called "double layer" arising from the difference in concentration of salts between the pore water and the fluid adjacent to the clay particles may cause a soil suction that is lumped into the matrix suction component, which is measured by the pressure membrane cell. The process of dilution of the pore water during diffusion into the free water may affect the double layer and may therefore change the matrix suction for a particular water content. This phenomenon may have occurred in the soil specimens described in Reference 43 and could have been responsible for the difference in behavior of specimens tested with free distilled water and soil solution.

67. The matrix soil suction pressure in the pore water of the specimen is measured for given applied vertical pressures, permitting simulation of in situ overburden pressures. The cell is designed for loading pressures P_o of at least 30 atm. The overburden pressure exerted on the specimen is the difference between the loading and extraction pressures. The extraction pressure or range of matrix suction is limited by the bubbling pressure of the ceramic plate to about 15 to 20 atm. The size of specimens accommodated in the cell is 4.25 in. in diameter by 0.5 in. high. Changes in the matrix suction may be related to changes in water content and specimen thickness.

68. Several calibration problems complicate analyses of results from pressure membrane tests. Percent changes in the vertical dimension $(\Delta H/H) \times 100$ may be less than percent volumetric changes $(\Delta V/V) \times 100$ because the perimeter of the specimen may pull away from the inside perimeter of the pressure cell when water is extracted from the specimen. Actual changes in void ratio may be as much as three times the observed change in height for isotropic specimens. Other calibration problems result from drying of the specimen by the applied air pressure, strains due to the air pressure exerted on the cell, and errors in volume of water expelled from or taken into the specimen due to accumulation of air bubbles beneath the ceramic plate.[43] Air from the applied air pressure diffuses through the specimen and plate in the pore water accumulating beneath the plate. Periodic flushing is required to remove this air. If the bubbling pressure of the ceramic plate is exceeded, air flows freely through the specimen and plate causing complete loss of control of water flowing into or from the specimen.

69. Time required to perform one complete cycle of a pressure membrane test from 0.1 to 15 atm and back to 0.1 atm was about 4 weeks, allowing 2 days for equilibration at each level of extraction pressure. Technician time was needed to cement a ceramic plate in the plate ring and test for air leaks, calibration of the cell, and monitoring. Equipment costs were about $3000 per cell including associated equipment such as valves, tubing, and pressure gages. Further details of the pressure membrane apparatus may be found in Reference 43.

Thermocouple Psychrometric Measurement of Suction

70. The thermocouple psychrometer measures the relative humidity in the soil by a technique referred to as Peltier cooling. By causing a current to flow through a single thermocouple junction in the proper direction, that particular junction will cool, causing water to condense on it when the dew point temperature is reached. Condensation of this water inhibits further cooling of the junction, and the voltage developed between the thermocouple and reference junctions is measured by a

microvoltmeter. Some examples of thermocouple psychrometers are shown in Figure 14.[52,53]

71. The relative humidity of the soil is related to total suction by the laws of thermodynamics

$$\tau^{o} = - \frac{RT}{V_{mv}} \ln \frac{p}{p_{o}} \qquad (27)$$

where

τ^{o} = total suction free of external pressure except atmospheric pressure, atm

R = universal gas constant (82.06 cc-atm/°K-mole)

T = absolute temperature, °K

V_{mv} = volume of a mole of liquid water (18.02 cc)

p/p_{o} = relative humidity

p = pressure of water vapor, atm

p_{o} = pressure of saturated water vapor, atm

The osmotic suction can be estimated by adding distilled water to the soil specimen and evaluating the total suction at high water contents (Figures 4 and 5).

72. Laboratory measurements to evaluate total suction may be made with the apparatus illustrated in Figure 15 and listed as follows:

Item	Quantity	1975 Cost
Psychrometric microvoltmeter (including cooling circuit)	1	$ 660
Switching circuit for 12 psychrometers	1	120
Thermocouple psychrometers	12	180
Dry ice chest with lid (insulated)	2	100
No. 13-1/2 rubber stoppers	12	20
Size 10 to 30°C thermometer	1	10
Metal pint paint cans with lids	24	10
	Total	$1100

The commercially available psychrometric microvoltmeter listed above includes a cooling circuit and it is different from that illustrated in

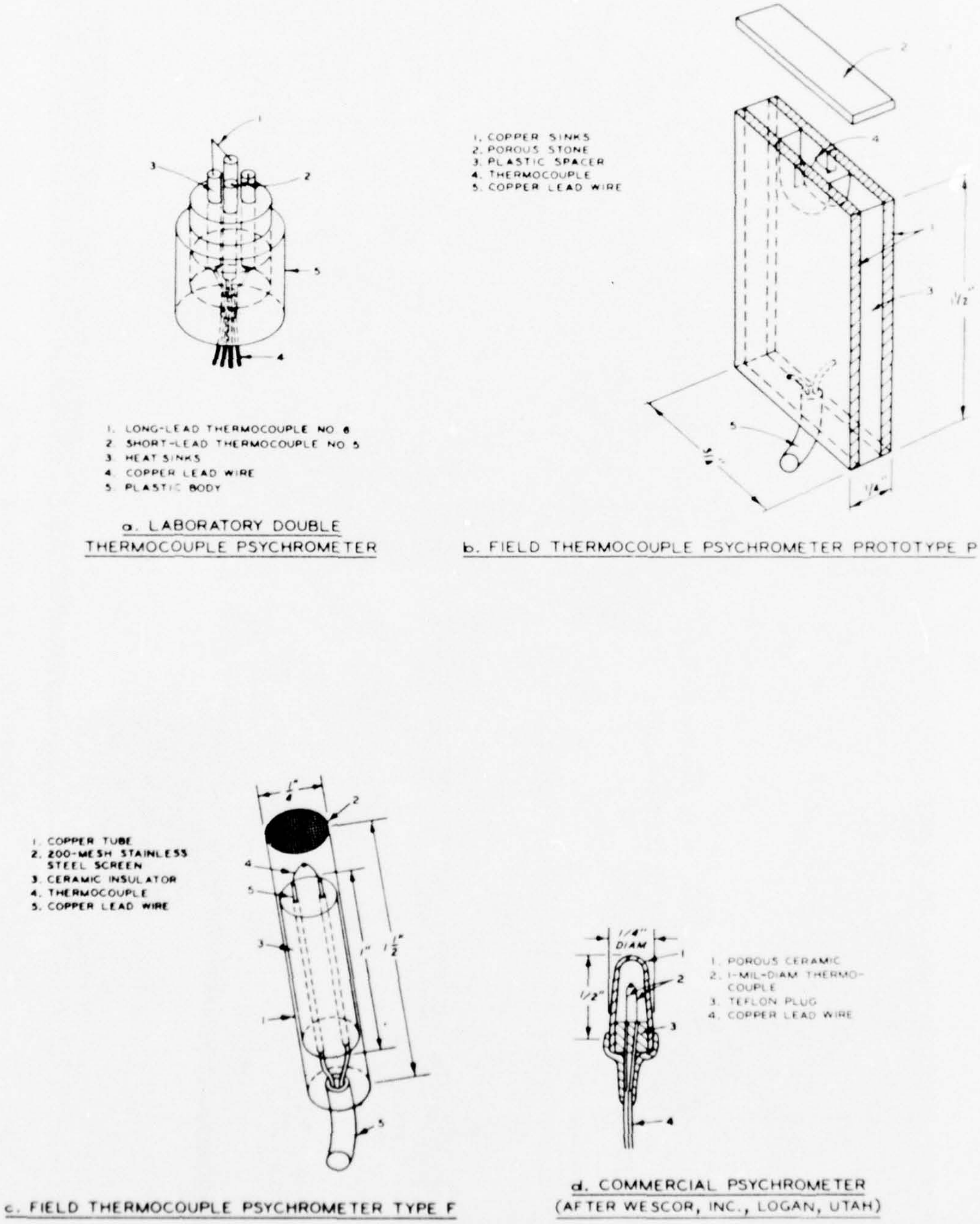

Figure 14. Laboratory and field thermocouple psychrometers

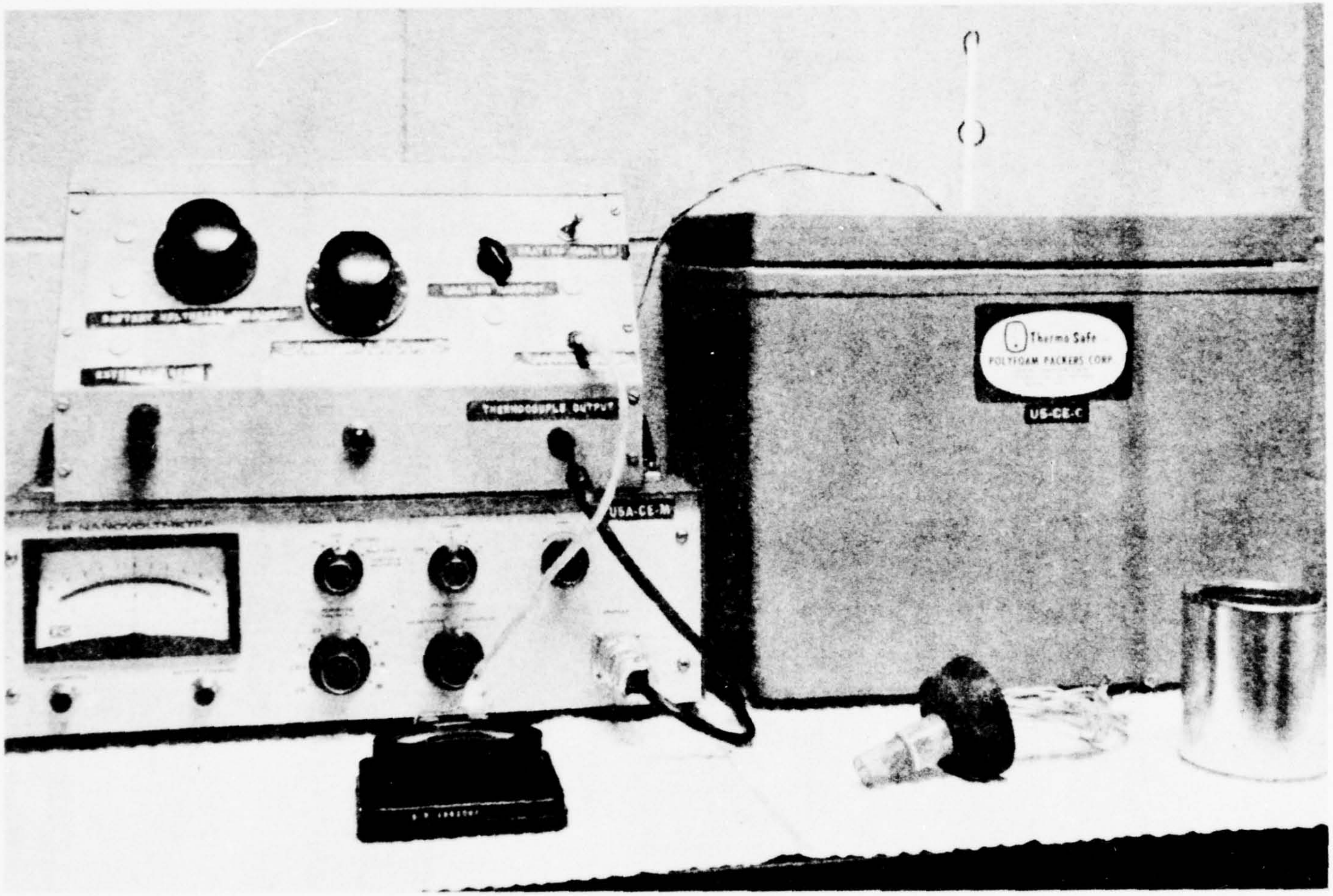

Figure 15. Monitoring system

Figures 15 and 16. The switching circuit can be assembled from commercially available parts (Figure 16). The above equipment permits evaluation in 2 days of the suction-water content relationship of soil based on 12 data points. Only two points are needed to obtain an approximate curve, if soluble salts are in low concentration, because of the linear relationship (Equation 3).

73. The thermocouple psychrometers should be calibrated prior to suction measurements with known concentrations of salt solutions to determine the microvolt-total suction relationship. The following concentrations of chemically pure potassium chloride (KCl) solution in distilled water were used for calibration:

Gram-Formula Weight per 1000 g Water, M	Grams of KCl per 1000 ml water	Relative Humidity Percent	Suction at 25°C, atm
0.05	3.728	99.83	2.3
0.20	14.91	99.36	8.8
0.50	37.27	98.42	21.6
1.00	74.55	96.84	43.4
2.00	149.10	93.68	88.5

The calibration curves of all six of the commercial (Wescor) psychrometers acquired for the subject study were similar and could be expressed by

$$\tau^{o} = 2.5E_{25} - 1.5 \tag{28}$$

where

τ^{o} = total suction, atm

E_{25} = microvolts at 25°C

The maximum attainable suction with the calibration solutions was 43.4 atm. The microvolt readings were usually taken at room temperatures from 20 to 25°C and were converted by[53,54]

$$E_{25} = \frac{E_t}{0.325 + 0.027t} \tag{29}$$

where E_t is the microvolt output at temperature t°C.

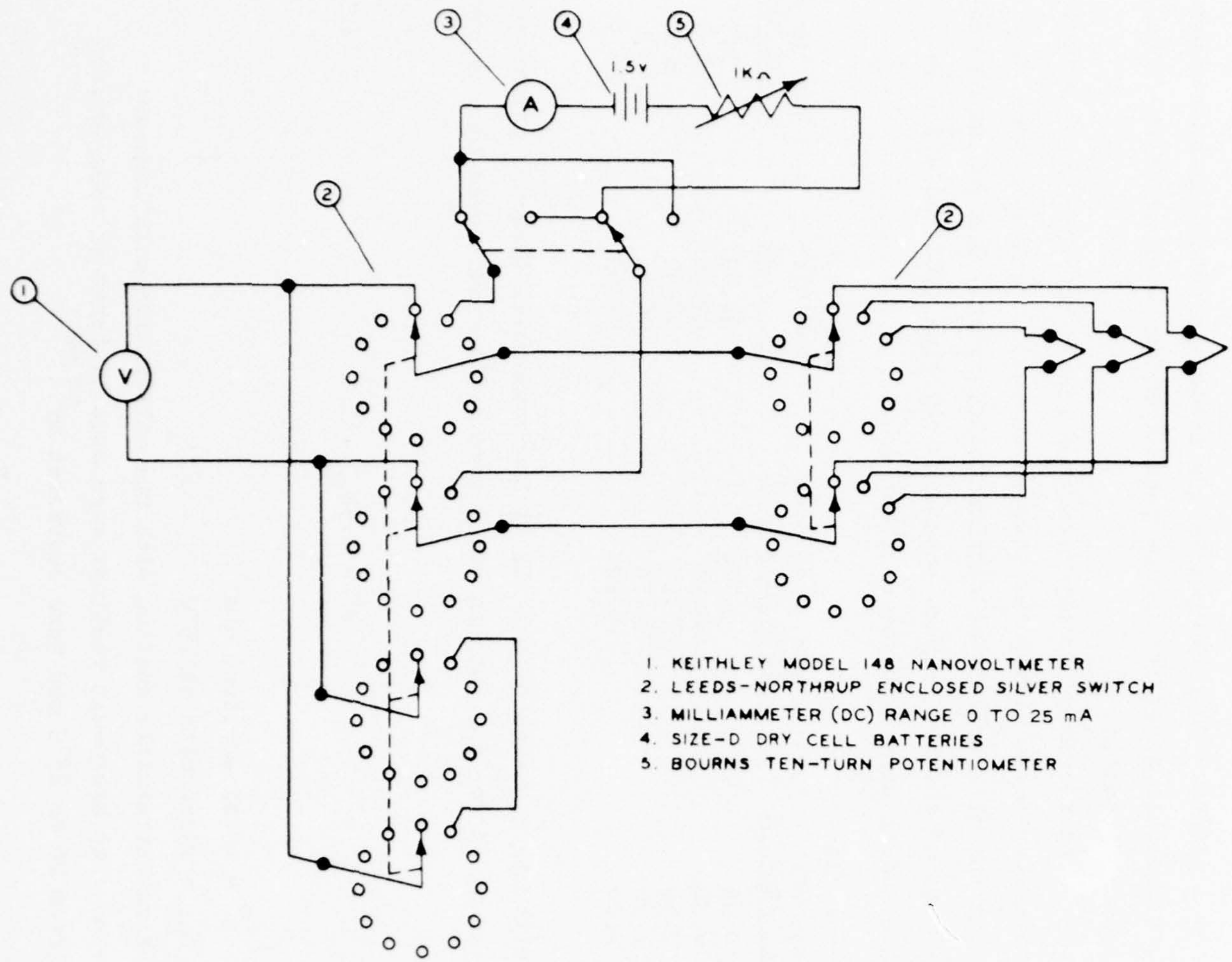

Figure 16. Electrical circuit for the thermocouple psychrometer

74. Specimens for testing were prepared by breaking 500 to 1000 g of undisturbed soil into 1- to 2-in. pieces and inserting them into six metal pint containers. Metal containers were necessary to reduce temperature gradients within the soil and the surrounding atmosphere. The containers need not be resistant to corrosion because iron oxide was found to have no discernable effect on the suction readings; however, rusting should not be permitted to cause holes in the container, which will destroy air tightness and lead to incorrect suction readings. Small amounts of distilled water on the order of 1 or 2 ml were added to some containers and sealed with lids for 48 hr. Other containers were left with lids off for different times up to 48 hr to permit drying.

75. The thermocouple psychrometers, following specimen preparation, were inserted into the pint containers with the specimens and the containers were sealed with No. 13-1/2 rubber stoppers. Equilibrium of the relative humidity in the air measured by the psychrometers inside the pint containers and the relative humidity in the soil specimens was obtained within 48 hr. Six containers were inserted into one 1- by 1- by 1.25-ft chest insulated with 1.5 in. of foamed polystyrene. The insulated chest promotes stable temperature conditions; temperature equilibrium was attained within a few hours after placing the cover. Following a 48-hr interval for equilibration of the relative humidity, a cooling current of 8 mA was applied for 15 sec. The microvoltmeter was immediately switched into the psychrometer circuit following cooling of the thermocouple junction. The maximum microvoltage obtained on the meter was recorded as the E_t of the soil specimen. Water contents of the specimens were obtained following the psychrometric voltage readings.

76. Half of the specimens of a particular soil were saved until the suction could be evaluated on the other half. This procedure permitted improved estimates of drying times and the amounts of water that should be added to the remaining specimens for obtaining optimum distribution of the 12 data points that determine the suction-water content relationship.

77. Total technician time for evaluation of a 12-point suction-water content relationship was about 1/2 day to set up the test, perform

readings, determine water contents, and evaluate the data. A straight line was drawn through the plotted data points that appeared to best represent the suction-water content relationship (Figure 4). The dry densities of the specimens were evaluated from the laboratory swell tests, which may have introduced some error in the suction swell pressures SP_s shown later. Better correlations of SP_s may be obtained by evaluating dry densities on soils prior to breaking the soils into pieces for the suction tests. Further details of evaluating total suction from thermocouple psychrometers are available in Reference 53.

PART IV: ANALYSIS OF RESULTS

78. Classification, swell, and suction tests were performed on a variety of undisturbed soil samples from various sources. Samples were obtained in the spring of 1973 from the Clinton installation of WES near Jackson, Miss.; the test pier site and dental clinic from LAFB, Tex.; Fort Sam Houston, Tex.; KAFB, Tex.; and Fort Carson, Colo. Brief descriptions of these soils and climates are given in Table 4. Boring logs of these samples with soil classification data taken from Table 5 are shown in Figures 17-20. Boring samples P1-5, P4-7, and P4-9 were taken in October 1973 from the test section in Fort Carson (Table 5) during construction of the test section to supplement samples from boring BOQ3 (Figure 17).

79. Laboratory tests were conducted to investigate the behavior of the soil suction parameters, compressibility factor, swell pressures, suction and swell indices, effect of confining pressure on suction, and volume change relationships. Comparisons of the swell behavior of the tested soils were made from analyses of several well known methods and the previously described new method.

Soil Suction Parameters

80. The soil suction parameters $\bar{A}$ and $\bar{B}$ may be evaluated directly from a plot of the suction data as illustrated in Figures 4 and 5. All of the $\bar{A}$ and $\bar{B}$ parameters shown in Table 6 were evaluated from straight lines on plots similar to Figures 4 and 5 that represent the best visual fit. Most soils did not indicate significant osmotic suction; hence, the total suction was essentially the matrix suction. For those soils for which the osmotic suction was significant, the osmotic suction was subtracted from the total suction to indicate a matrix suction-water content relationship, from which the $\bar{A}$ and $\bar{B}$ parameters were evaluated. A study of the effect of osmotic suction on swell behavior was not included as part of this study. Rough estimates of $\bar{A}$ and $\bar{B}$ may be made by assuming that the parameters are functions

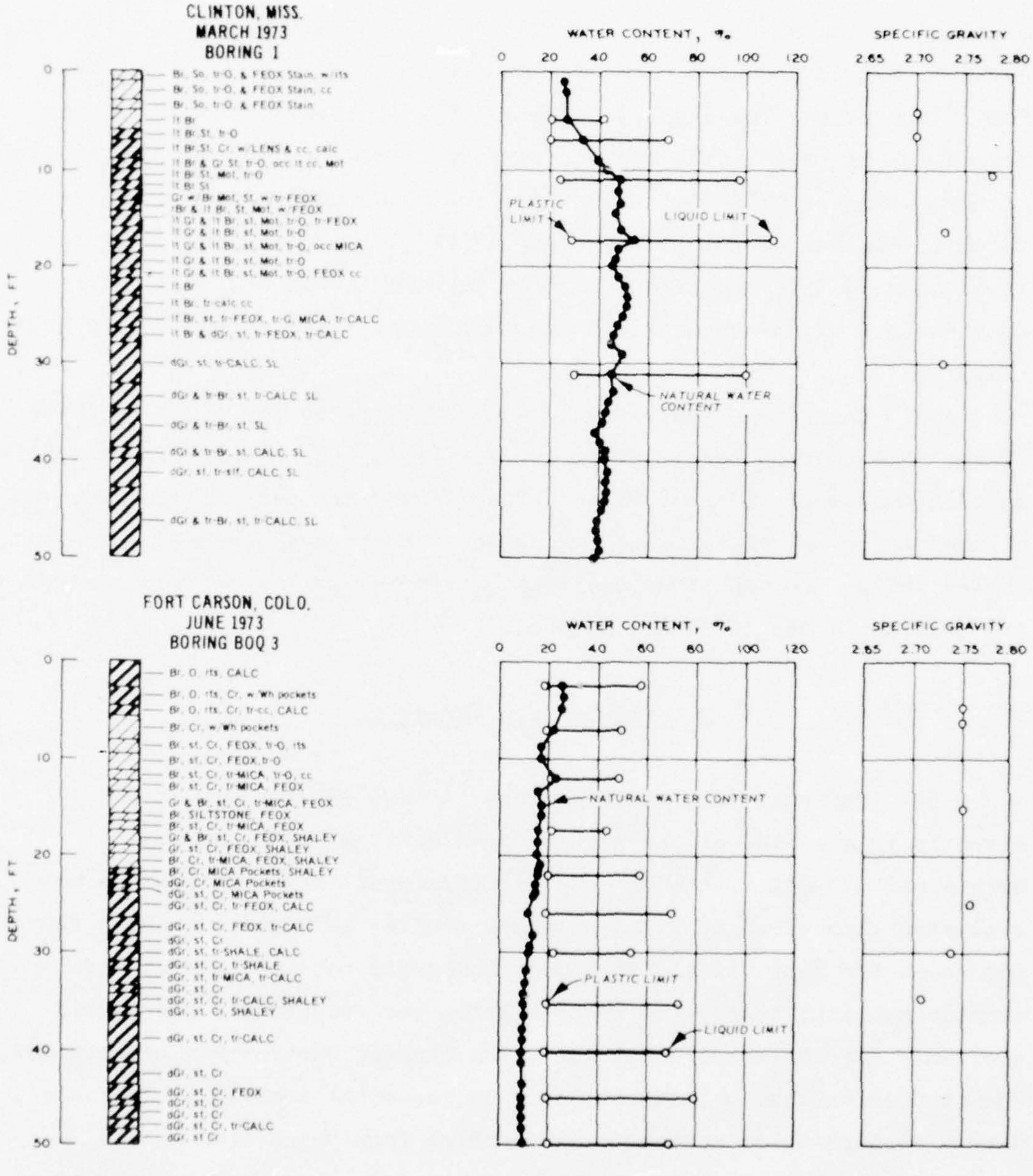

Figure 17. Boring logs from Clinton, Miss., and Fort Carson, Colo.

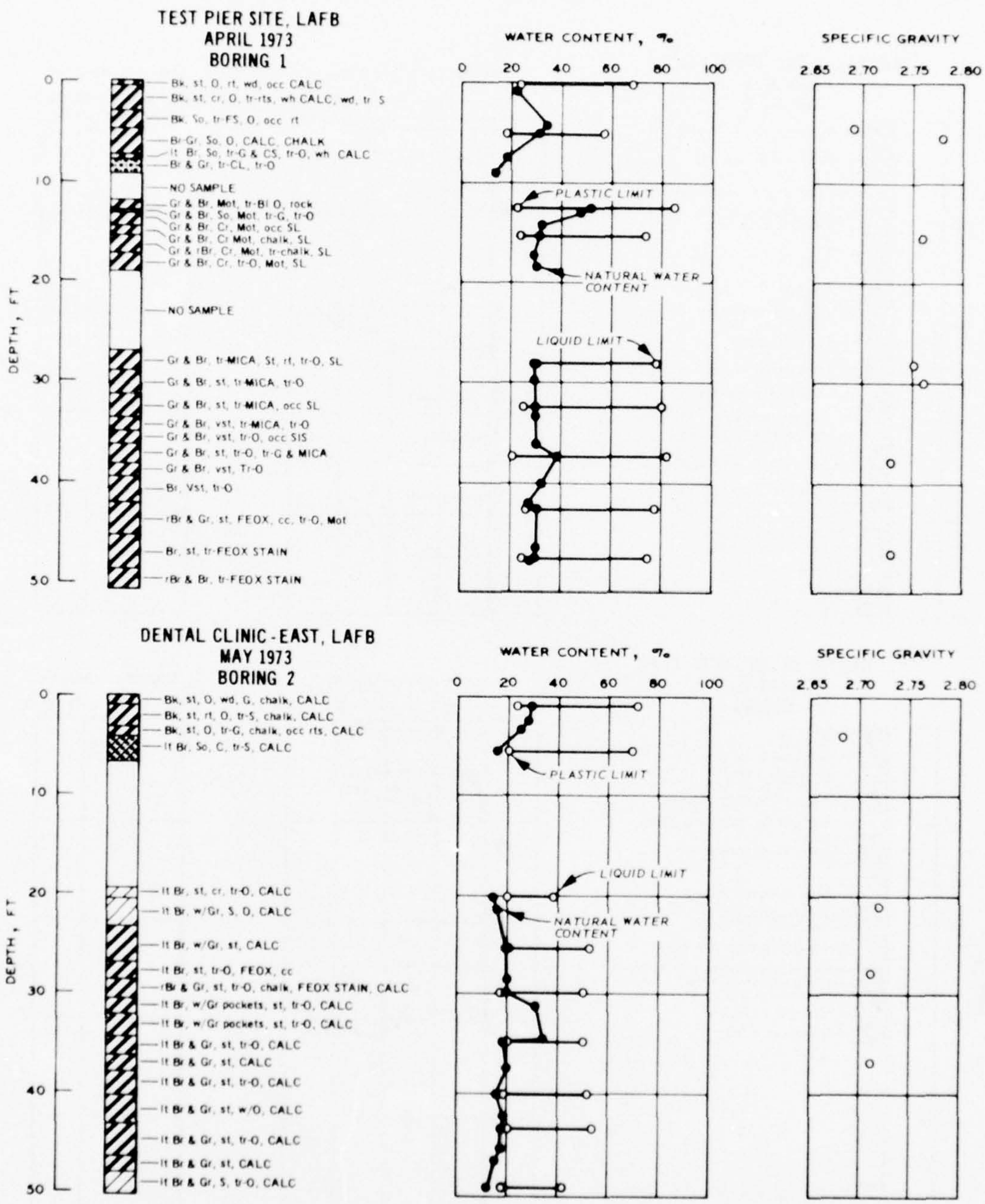

Figure 18. Boring logs 1 and 2 from LAFB, Tex.

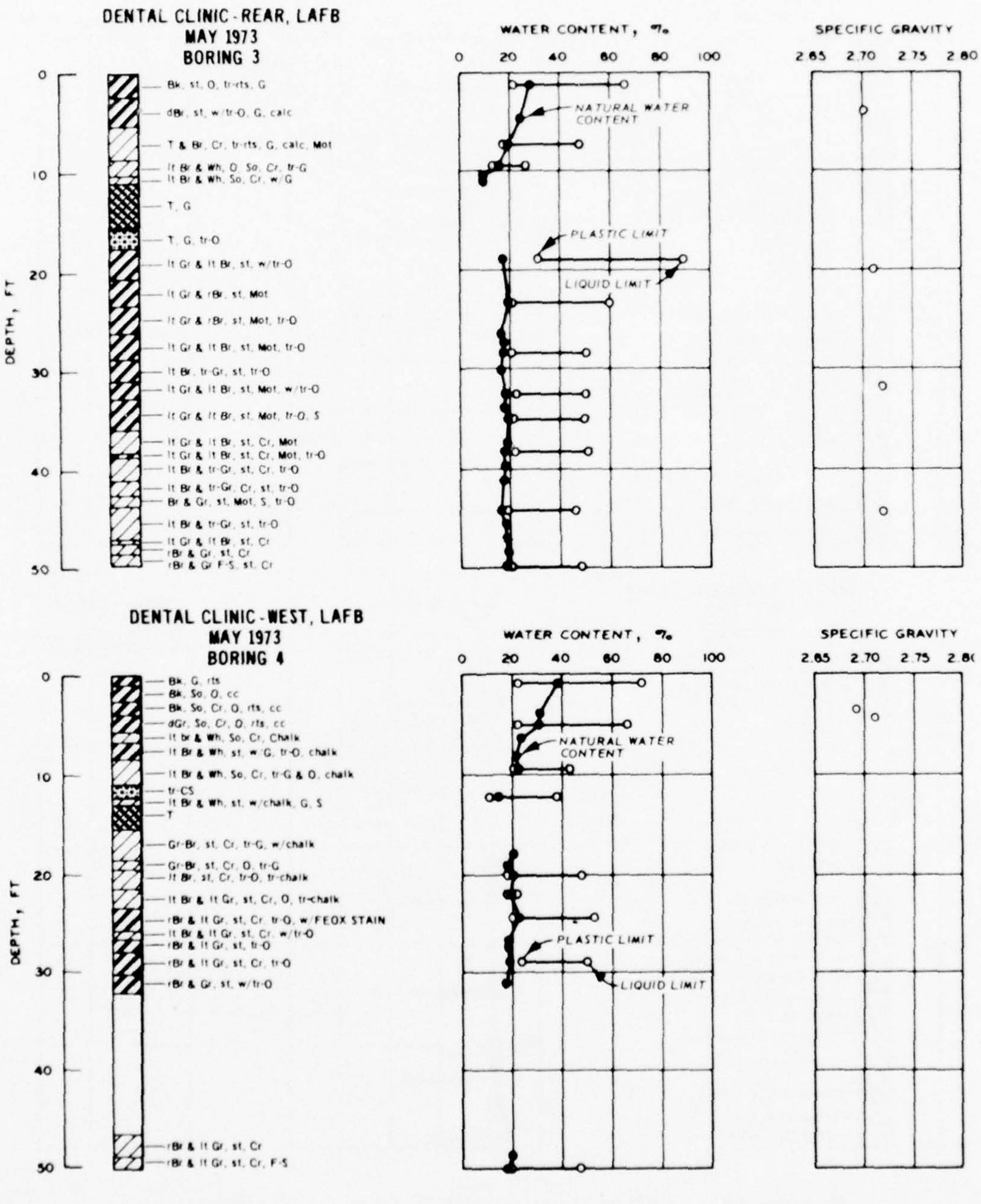

Figure 19. Boring logs 3 and 4 from LAFB, Tex.

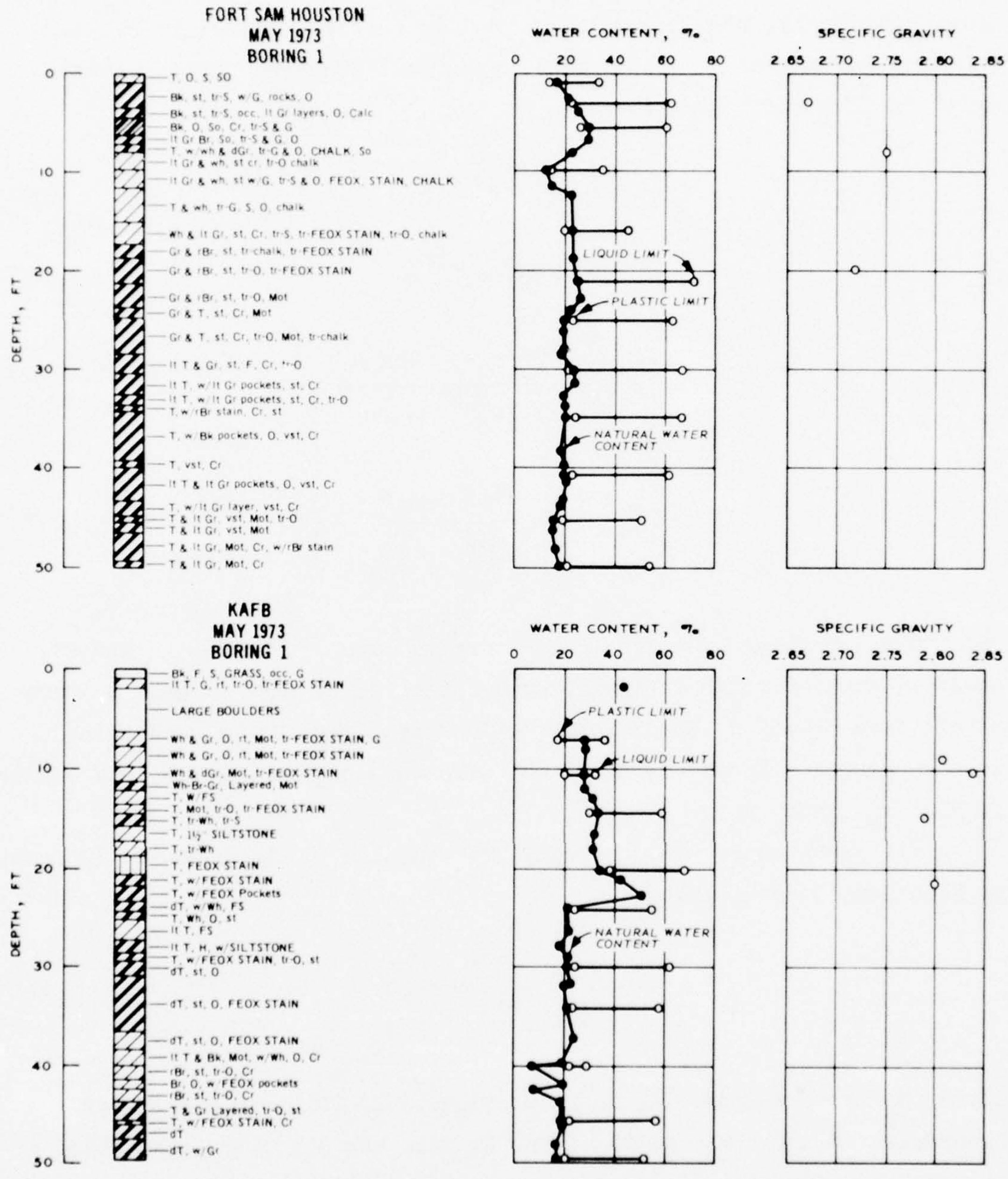

Figure 20. Boring logs from Fort Sam Houston and KAFB, Tex.

of the PL and e_o or w_o (Table 5). Figure 21 shows that $\bar{A} - \bar{B}PL$ plotted against the logarithm of e_o or w_o yields a linear relationship. Similarly, the logarithm of $\bar{A} \div \bar{B}$ may be empirically related to the logarithm of e_o and w_o (Figure 22). Solving these relationships for the suction parameters,

$$\bar{A} = 1.7w_o^{0.85}\bar{B} \qquad (30)$$

or

$$\bar{A} = 35.9e_o^{0.85}\bar{B} \qquad (31)$$

$$\bar{B} = \frac{-8.33 + 6.5 \log w_o}{1.7w_o^{0.85} - PL} \qquad (32)$$

or

$$\bar{B} = \frac{1.7 + 6.5 \log e_o}{35.9e_o^{0.85} - PL} \qquad (33)$$

for soils investigated herein. The correlations of Figures 21 and 22, however, are too rough to permit good estimates of the $\bar{A}$ and $\bar{B}$ parameters from Equations 30-33 for practical applications in predicting volume changes. Equations 30-33 may also become negative for some values of PL, e_o, and w_o.

81. Setting Equation 30 equal to Equation 31 and Equation 32 equal to Equation 33 shows that

$$\frac{100e_o}{w_o} = \frac{G_s}{S} \approx 2.8 \qquad (34)$$

Since G_s varies from 2.67 to 2.78 (Figures 17-20), the degrees of saturation S of these soils on the average are slightly less than one at natural water content under zero confining pressure. Many of these soils were taken from strata with positive pore water pressures measured in field piezometers such that S should equal one at the field confining pressure.

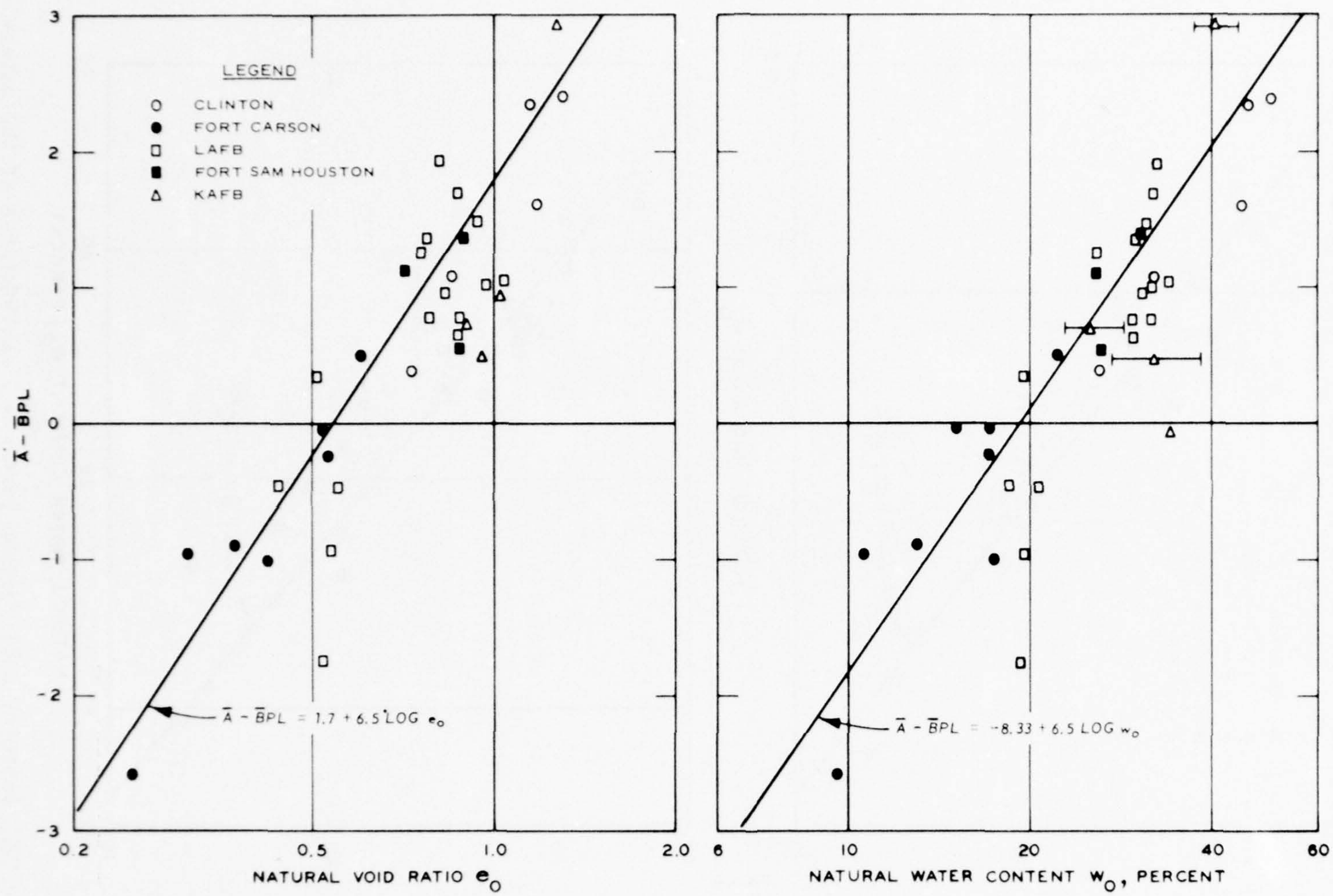

Figure 21. Soil suction parameters as functions of the PL, natural void ratio, and natural water content

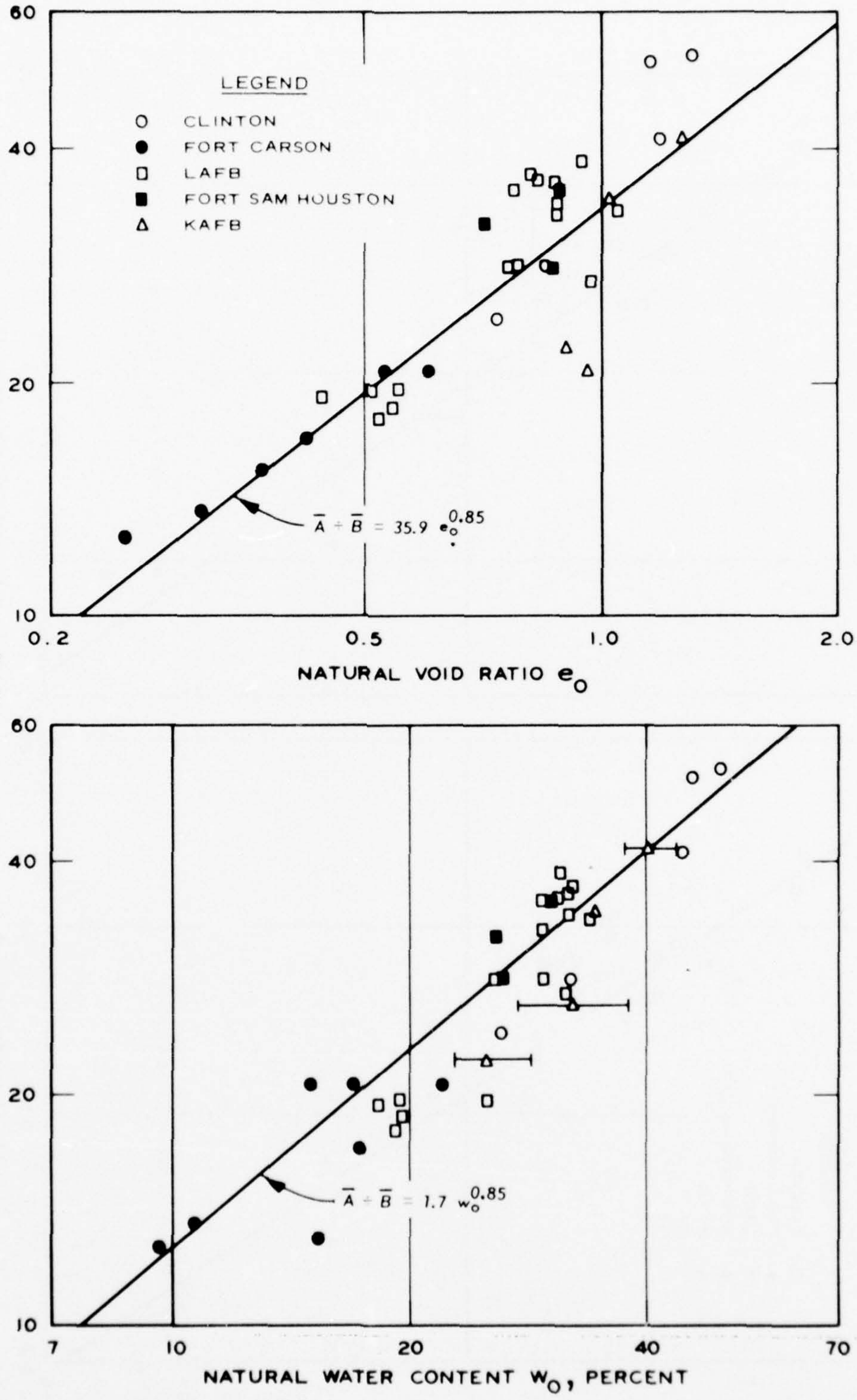

Figure 22. Ratio of soil suction parameters as functions of the natura void ratio and natural water content

The Compressibility Factor

82. The compressibility factor is measured as the slope of the specific total volume plotted against the water content. The specific total volume versus water content relationships of undisturbed soils from Fort Carson (Figure 23) were obtained from test results of the pressure membrane cells. The changes in specific total volume were computed from the measured change in height of the specimens and do not include lateral changes in dimensions. The overburden pressure σ_{vo} was varied for specimens taken from sample BOQ3-4 (5.7 to 7.0 ft) and sample BOQ3-20 (24.7 to 26.0 ft) to study the effect of surcharge loading on the compressibility factor.

83. The slopes of the curves in Figure 23 are approximately linear for the range of water contents. Slopes are also approximately linear for soils from the Clinton (Figure 21, Reference 43) and LAFB (Figure 22, Reference 43) test sections. The curves for each of the samples BOQ3-4 and BOQ3-20 at different surcharge pressures (Figure 23) are approximately parallel; consequently, surcharge pressures do not appear to significantly influence the slope of these curves within the range of 0.40 to 2.25 tsf for the Fort Carson soils, which is the usual range of pressures for many tested foundation soils. The compressibility factor for vertical dimensional change is essentially independent of surcharge pressure for at least some soils.

84. The vertical compressibility factor $\overline{\alpha}$ for the Fort Carson soils determined from Figure 23 and the $\overline{\alpha}$ of soils from Clinton and LAFB (Table 7) are plotted as a function of the PI in Figure 24. The approximate linear relationship can be described as

$$\begin{aligned} PI < 5 \quad & \overline{\alpha} = 0 \\ 5 \leq PI \leq 100 \quad & \overline{\alpha} = 0.0105PI - 0.05 \\ PI > 100 \quad & \overline{\alpha} = 1 \end{aligned} \tag{35}$$

The compressibility factor for vertical dimensional change $\overline{\alpha}$ is about one half of the volumetric compressibility factor α (Equation 15) from Reference 48.

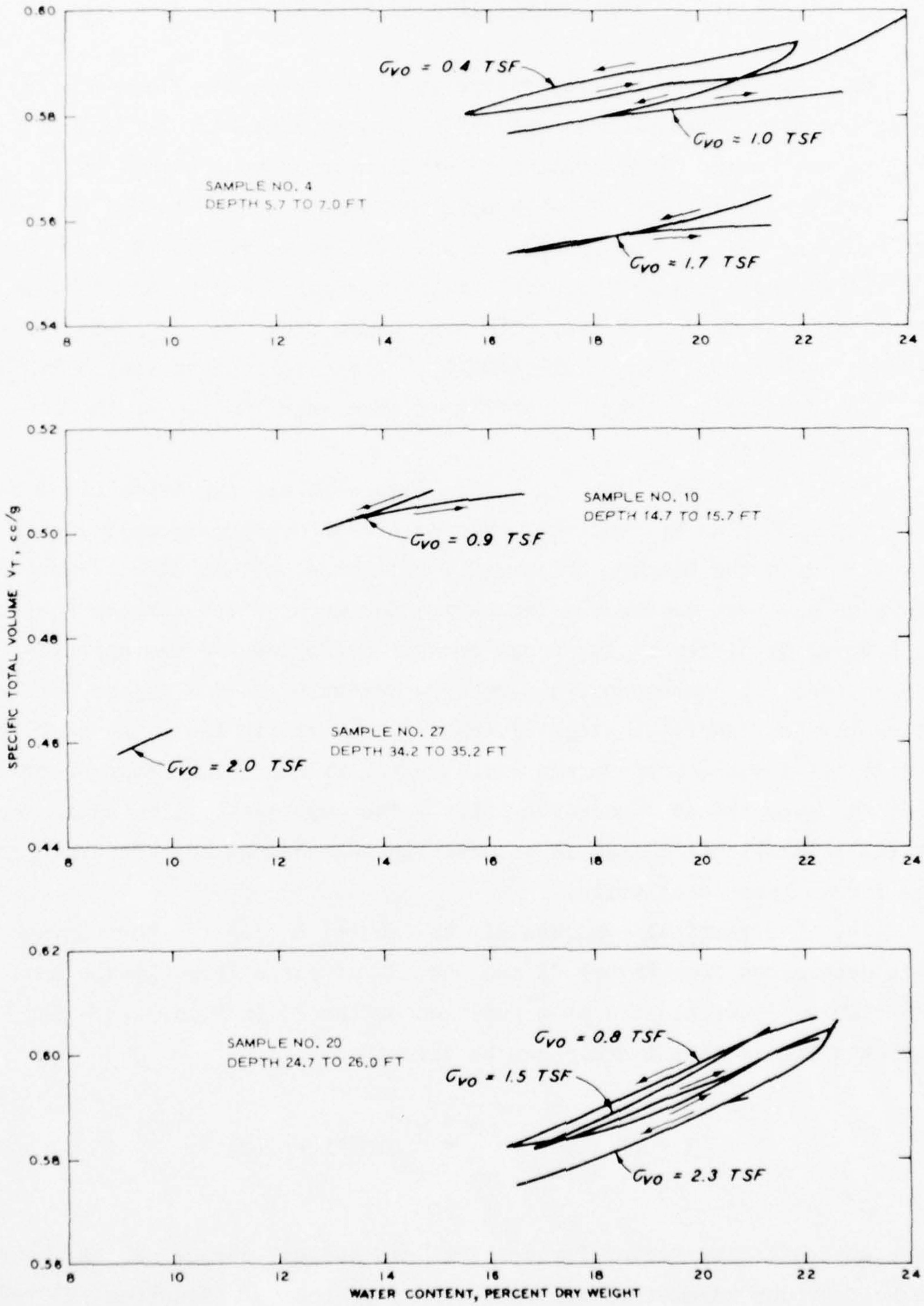

Figure 23. Shrinkage and swell curves for samples from boring BOQ3, Fort Carson

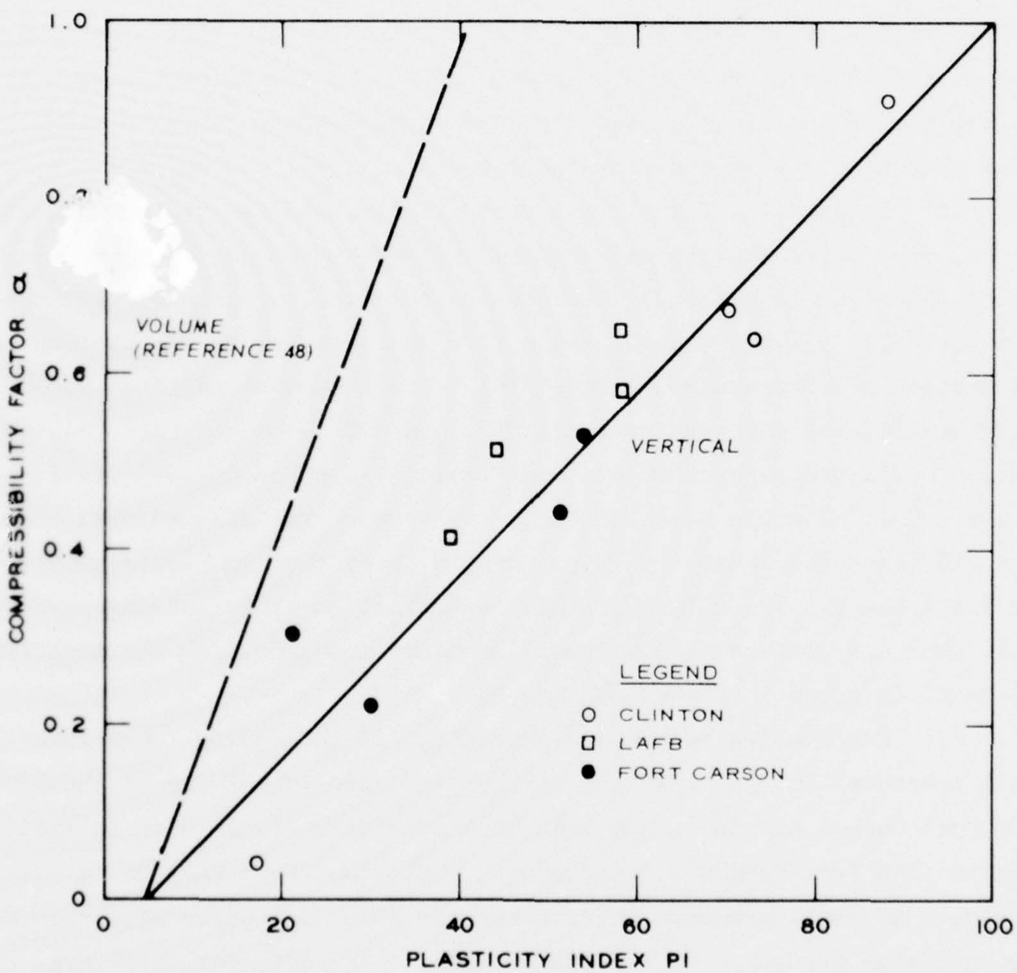

Figure 24. The compressibility factor as a function of PI

Comparison of Swell Pressures

85. Comparisons of the initial suction (Equation 3) with the suction swell pressure (Equation 19) in Table 8 show that these values are usually close. The τ^o_{mo} should be greater than SP_s , which is observed for more than half of the undisturbed soils. Calculations of SP_s greater than τ^o_{mo} are attributed to experimental errors, primarily in void ratio. Void ratios were only determined on the specimens that were tested in the laboratory CS tests and not on the suction specimens. The smaller values of τ^o_{mo} and SP_s were used for comparisons with swell pressures determined from the laboratory swell tests.

86. Comparison of swell pressures (Table 9) shows that the multiple SO tests often provide measurements of swell pressure greater than initial suctions (Equation 3) and the suction swell pressures calculated from Equation 19, while the SP tests give measurements much less than the suction swell pressure (Figure 25a). The remaining modified SO, modified CVS, maximum past pressure CVS, and ISO tests at least tend to provide swell pressures that are comparable with suction swell pressures (Figure 25b), although much scatter is apparent. The maximum past pressure CVS test consolidates the specimen in an attempt to reduce specimen disturbance. The effect of this consolidation is to increase the swell pressure compared to the swell pressure measured from the modified CVS test, as shown in Figure 25b and Table 9.

87. The suction tests were successful in indicating comparable swell pressures for the hard and highly overconsolidated Pierre shale from Fort Carson and the sandy lean clay from KAFB (Table 9). In fact, the specimen from KAFB No. 5 collapsed, while that from KAFB No. 4 developed no swell pressure after water was added following placement of the surcharge pressure σ_{vo} . The suction tests indicated that very little swell pressure should develop for these specimens from KAFB.

88. The scatter in the limited results (Figure 25) emphasizes the need of performing large numbers of tests using reliable and uniform test procedures if worthwhile data are to be obtained. The multiple SO and SP tests appear to be the least acceptable. The suction test is

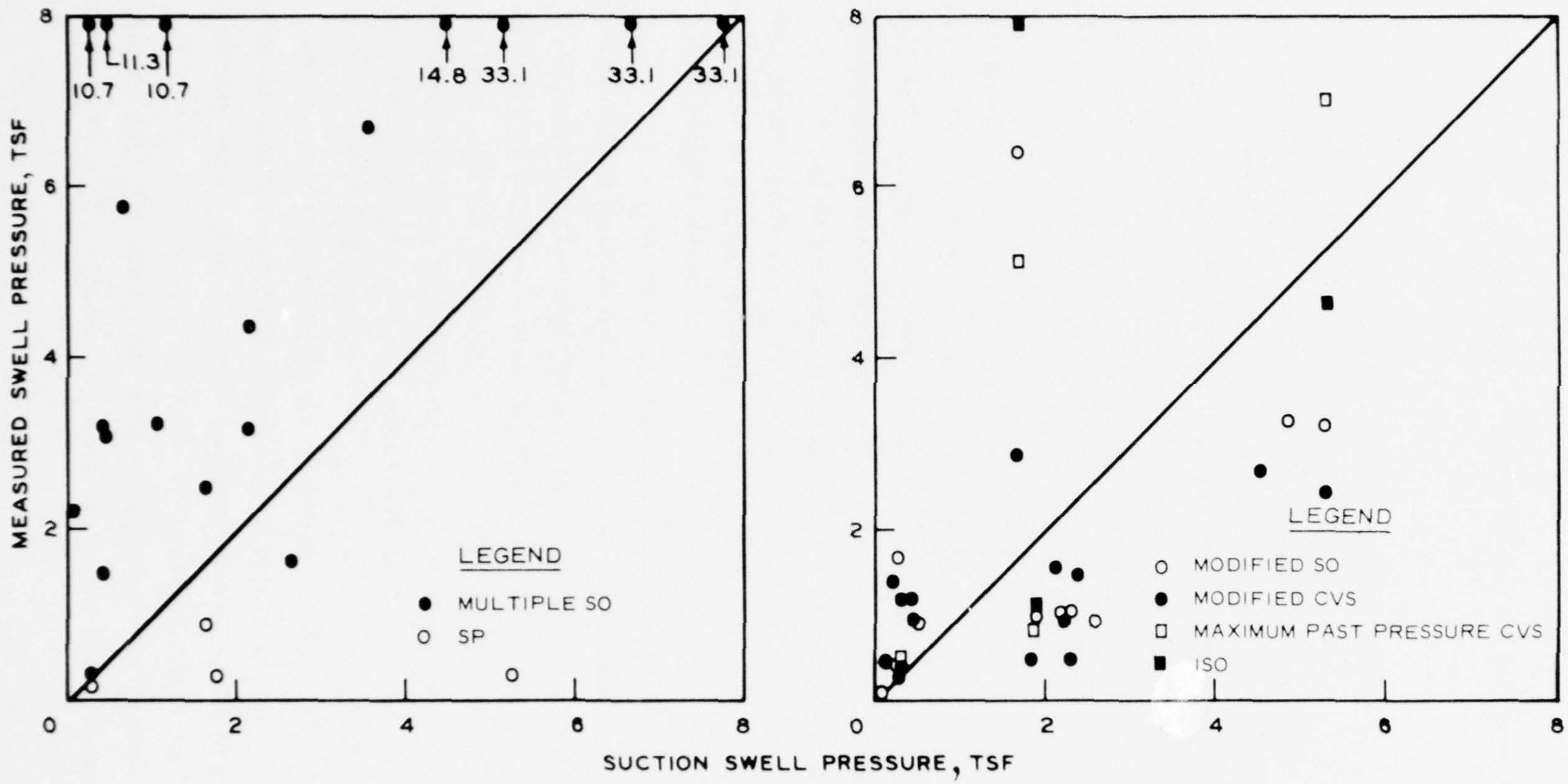

Figure 25. Comparisons of swell pressure

substantially easier and faster to perform than any of the other swell tests.

Comparison of Suction and Swell Indices

89. Index comparisons (Table 10) show that the suction index C_{τ} is from two to six times larger than the swell index C_s determined from the rebound curve of CS tests, depending on the type of swell test. The suction index based on volumetric compressibility $C_{\tau\alpha}$ is about six times greater than C_s from multiple SO and SP tests and only about four times greater than C_s determined from modified SO, modified CVS, maximum past pressure CVS, and ISO tests (Figure 26). The C_s evaluated from the ISO test is taken from the slope of the rebound curve from a surcharge pressure of 16 tsf, while the C_s from the other tests is the slope of the rebound curve from the swell pressure. The ISO swell index is about the same as $C_{\tau\bar{\alpha}}$ for the four tests completed using the ISO procedure (Figure 27). The suction index based on the vertical dimensional change compressibility factor $C_{\tau\bar{\alpha}}$ is about three times greater than the C_s from multiple SO and SP tests and only about two times greater than the C_s from other swell tests (Figure 27). The $C_{\tau\alpha}$ is therefore about two times greater than $C_{\tau\bar{\alpha}}$.

90. The compression index C_c measured from a standard consolidation test performed to evaluate the maximum past pressure is somewhat larger than $C_{\tau\alpha}$, while C_c from ISO tests tends to be somewhat less than $C_{\tau\alpha}$ (Figure 28). The C_c from ISO tests was measured on the slope of the consolidation curve near surcharge pressures equal to the swell pressure or surcharge pressure needed to reduce the void ratio to the initial void ratio, while C_c from consolidation tests to evaluate the maximum past pressure represents virgin consolidation. The C_c from the ISO test is basically the index used to evaluate in situ heave by Firth's procedure, which Jennings et al.[27] have found satisfactory for predicting field heaves.

91. Comparisons between indices show that $C_{\tau\bar{\alpha}}$ is usually greater than C_s and much less than C_c, while $C_{\tau\alpha}$ is close but

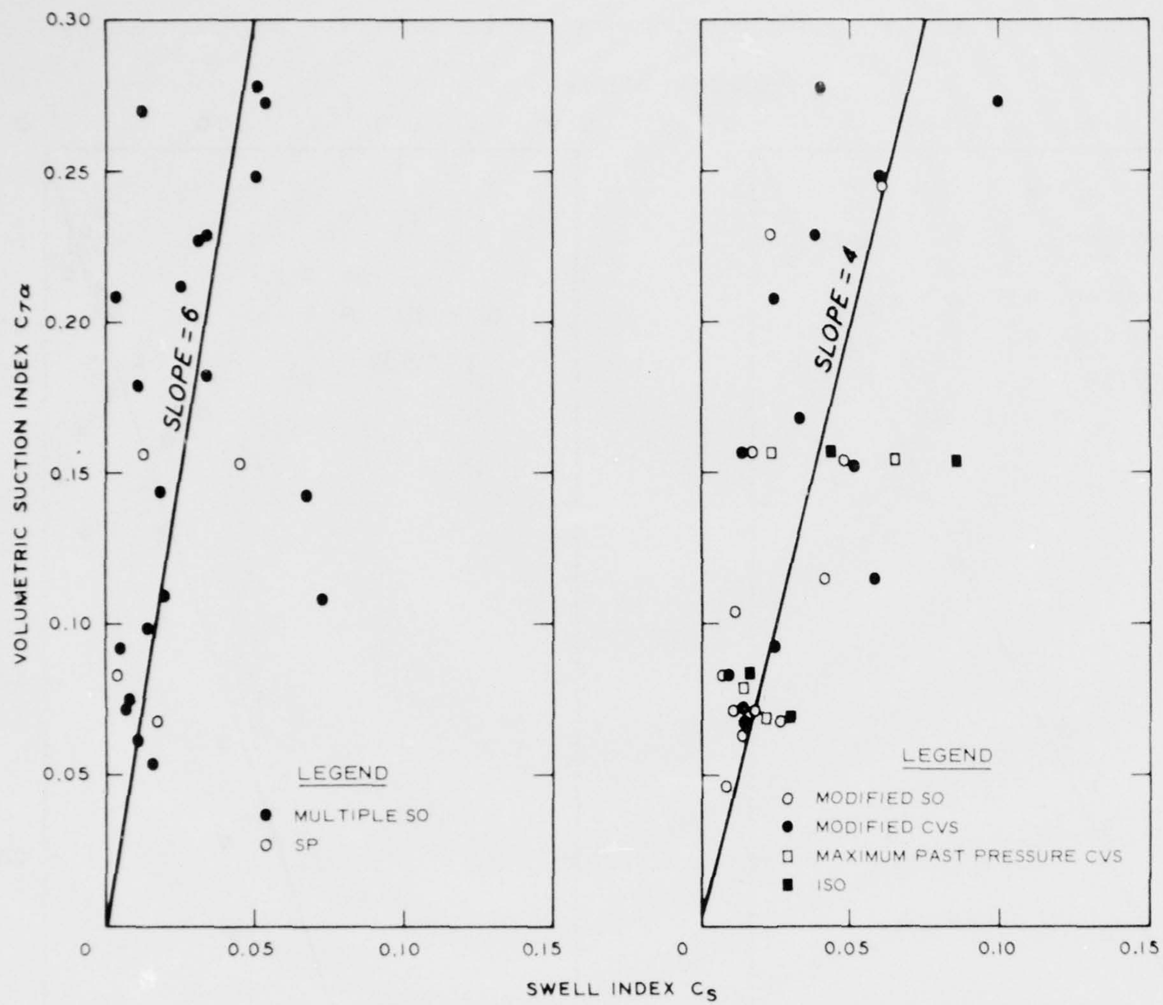

Figure 26. Comparisons of volumetric suction index with swell index

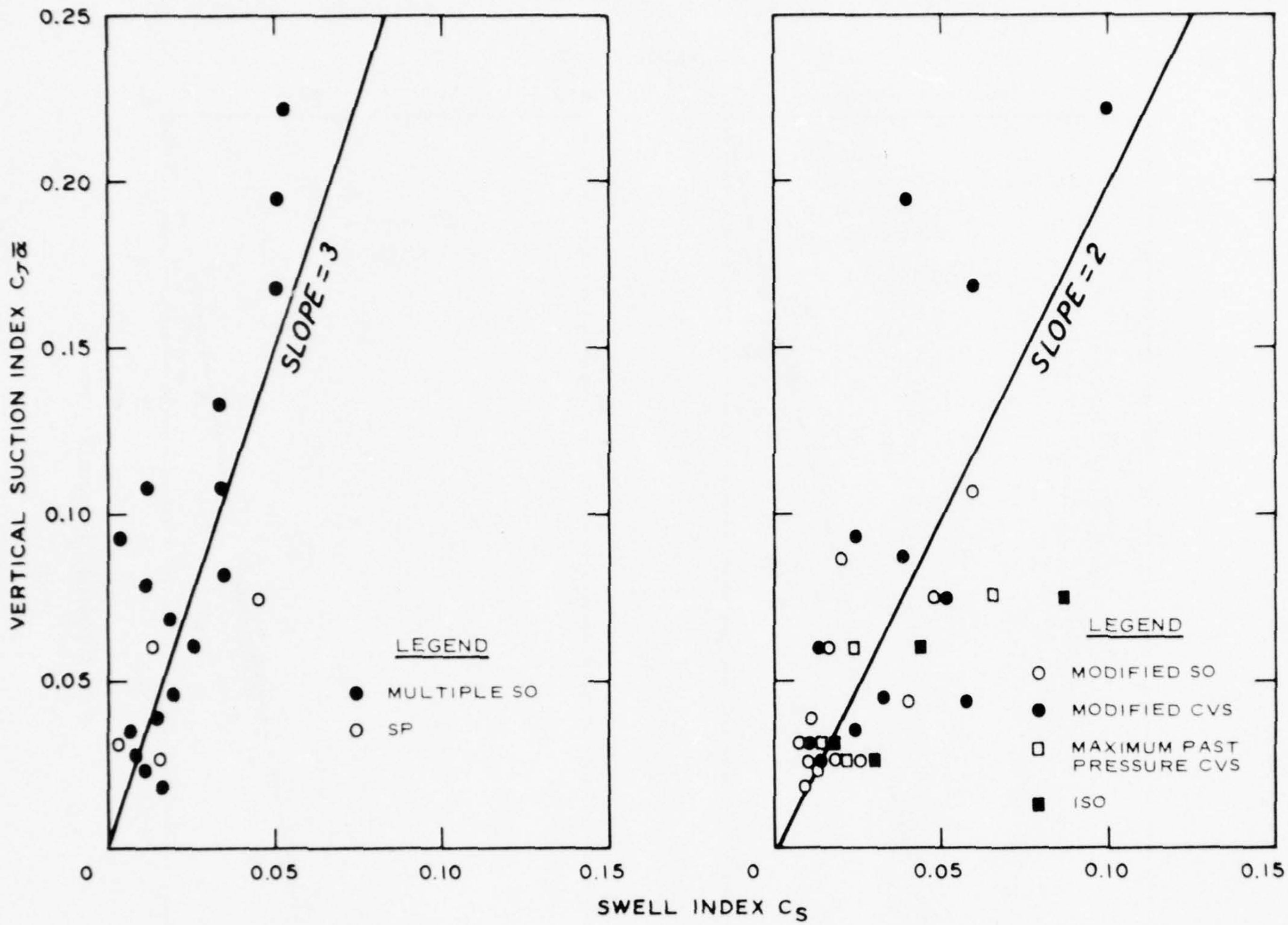

Figure 27. Comparisons of the vertical suction index with the swell index

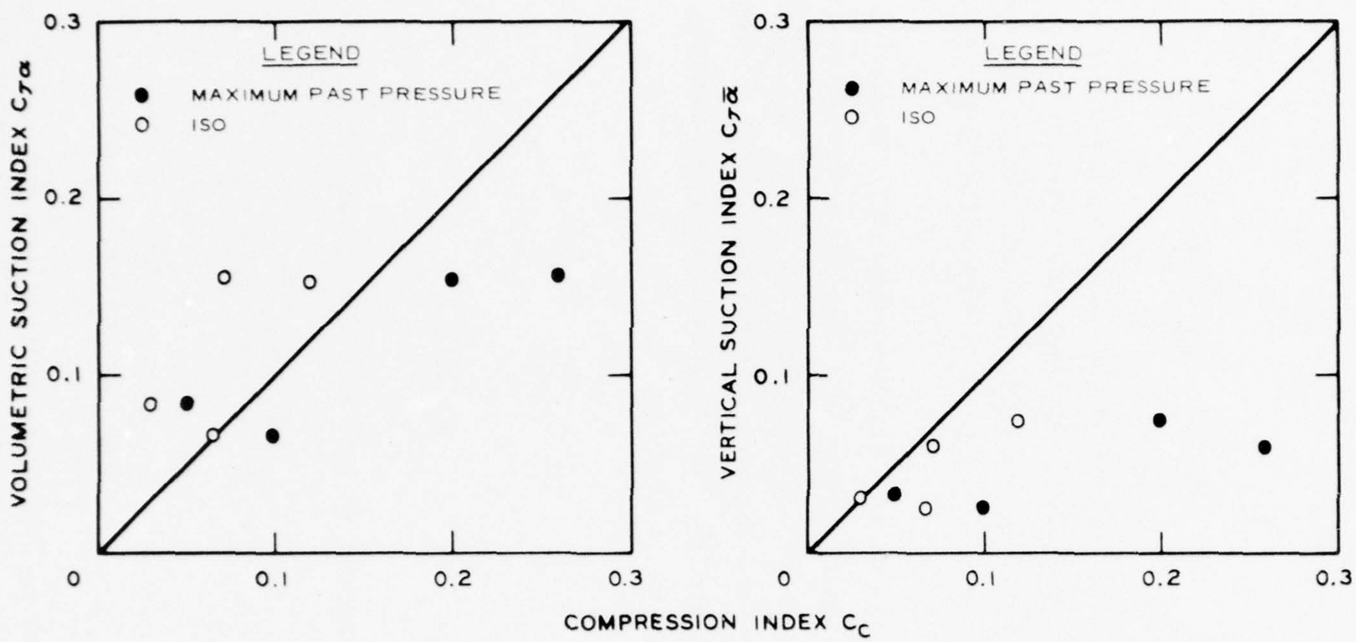

Figure 28. Comparisons of the suction indices with the compression index

still less than the C_c of virgin compression. The volumetric suction index with α set equal to one might be used as an estimate of the compression index. The relationships between the various indices may be expressed as

$$C_c > C_{\tau\alpha} > C_{\tau\bar{\alpha}} > C_s \tag{36}$$

All of the swell tests leading to the relative behavior between indices expressed by Equation 36 were relatively short-term. Allowing lengthy times of many months for swell during rebound down to the seating load for each decrement of load may lead to substantially larger values for C_s .

92. The PI increases with increasing swell or suction indices (Figure 29). All swell indices C_s from the different swell tests are included in Figure 29. The slope of PI versus C_s is about two times that of $C_{\tau\bar{\alpha}}$ and about four times that of $C_{\tau\alpha}$. These relationships support Equation 36. The Equation of the relationship between PI and $C_{\tau\alpha}$ in Figure 29 is given by

$$C_{\tau\alpha} = \frac{\alpha G_s}{100\bar{B}} = \frac{PI - 5}{260} \tag{37}$$

or

$$\bar{B} = \frac{2.6\alpha G_s}{PI - 5} \tag{38}$$

Substitution of Equation 38 into Equations 30 and 31 yields

$$\bar{A} = \frac{4.5\alpha G_s w_o^{0.85}}{PI - 5} \tag{39}$$

or

$$\bar{A} = \frac{93.3\alpha G_s e_o^{0.85}}{PI - 5} \tag{40}$$

Equations 37-40 may be applied to roughly estimate the $\bar{A}$ and $\bar{B}$ parameters. These equations will not lead to negative $\bar{A}$ and $\bar{B}$ parameters as easily as Equations 30-33.

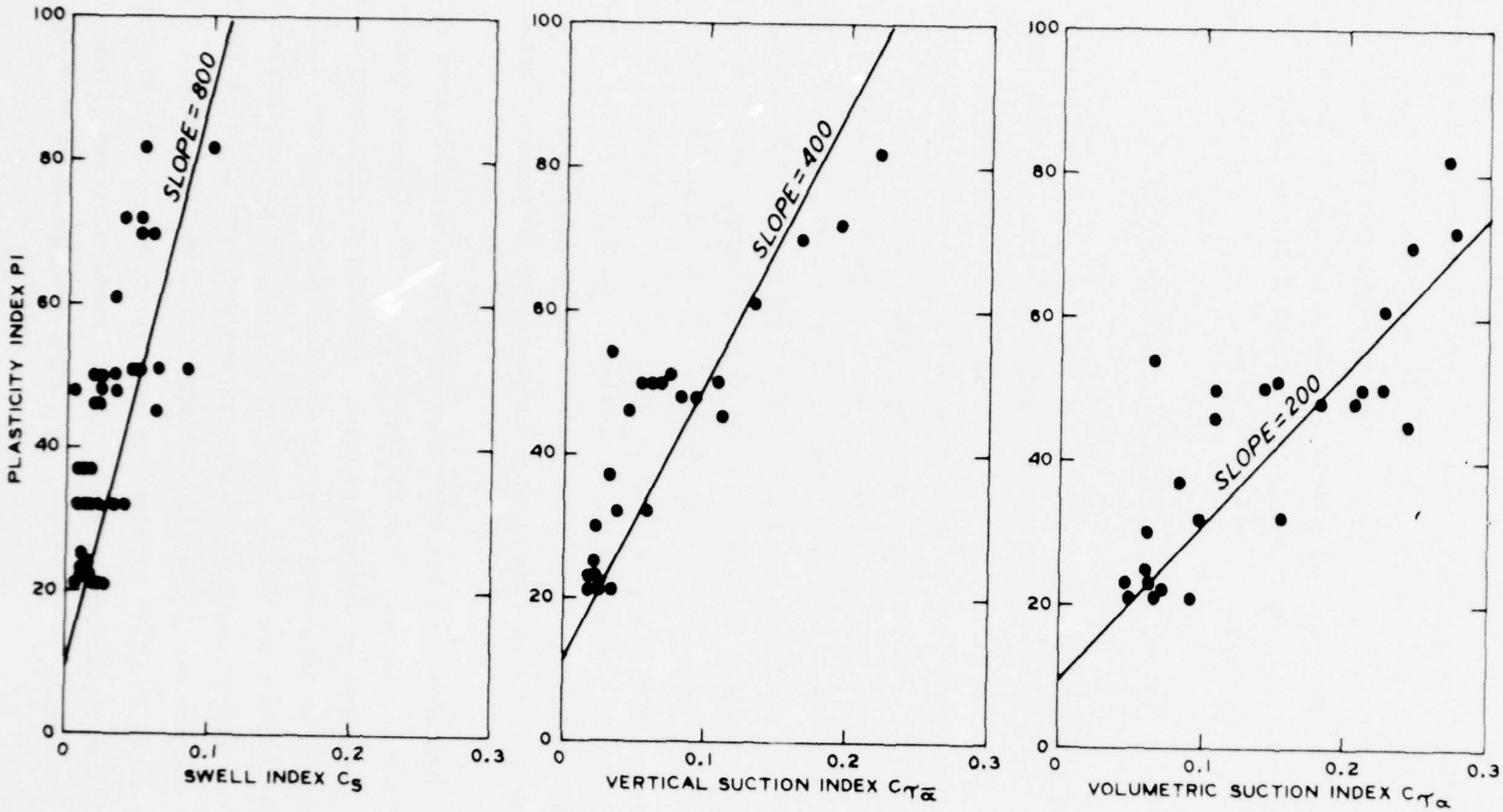

Figure 29. Comparisons of plasticity, swell, and suction indices

93. Substitution of $C_{\tau\bar{\alpha}}$ in Equation 26 may be more appropriate for fissured soils where lateral heave may occur and reduce the volume of fissures. $C_{\tau\alpha}$ may be better for intact soils in which all volume change occurs in the vertical direction. Relative magnitudes of suction indices are used later as a method for indentifying the relative degree of expansion of the soil.

Effect of Confining Pressure on Suction

Effect of vertical pressure

94. The effects of surcharge pressure σ_v on the Fort Carson soils are illustrated in Figure 30 for pressure membrane tests. The characteristic hysteresis is reduced in the second and third drying and wetting cycles of the soil specimen from sample BOQ3-4 (5.7 to 7.0 ft). These latter cycles are loaded beyond the in situ vertical pressure σ_{vo} of 0.4 tsf ft. There was some slight reduction in volume for σ_v of 1.0 tsf and a considerable reduction in volume at 1.7 tsf (Figure 23), but the suction-water content relationships following cycle one at 1.0 and 1.7 tsf are very similar (Figure 30). The total in situ suction curve computed from Equation 4 for a σ_{vo} of 0.4 tsf from measurements with thermocouple psychrometers is consistent with the pressure membrane curve. The difference in suction between the total and matrix suction is presumably an osmotic suction or is caused by differences between specimens.

95. Hysteresis in the soil specimen of Pierre shale from BOQ3-20 (24.7 to 26.0 ft) is small and differences in slope between cycles at different surcharge pressures are small. The total suction-water content curve may be displaced from the pressure membrane curve due to differences between specimens. Comparisons of the curves between the results from thermocouple psychrometers and pressure membrane apparatus generally suggest small osmotic suctions, which would be expected from the small total suctions observed at high water contents in Figure 4.

96. The slopes of the total in situ suction curves (Figure 30) are less than those of corresponding curves from pressure membrane tests.

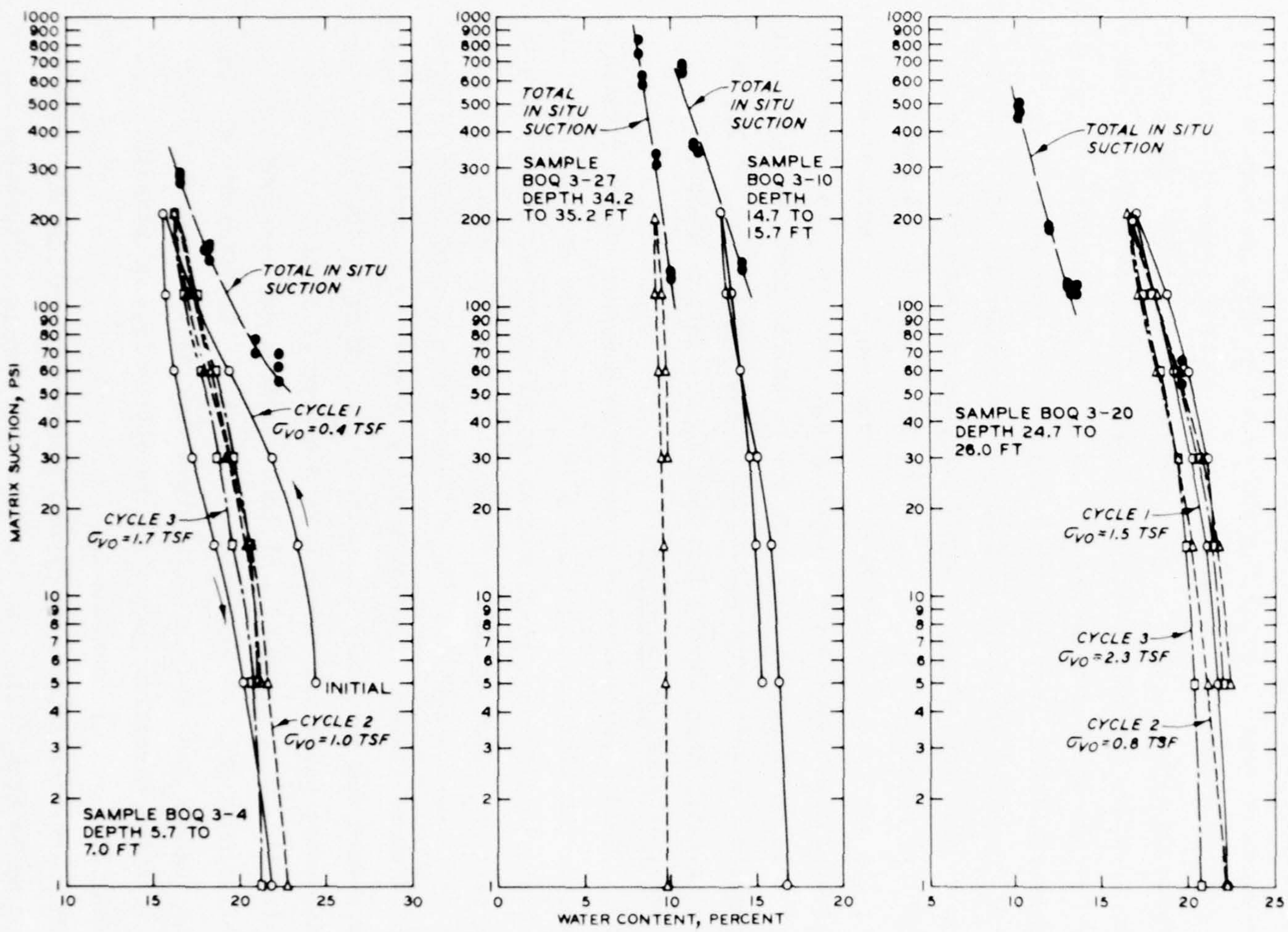

Figure 30. Soil suction-water content relationships of Fort Carson soil

The difference in slopes is primarily attributed to the higher suction levels determined from thermocouple psychrometers compared to the lower suction range of the pressure membrane apparatus. The characteristic suction-water content behavior for the pressure membrane apparatus is in the form of a reduction in slope at high suction levels followed by an increase in slope at still higher suctions (Figure 6). The results from the pressure membrane tests do not generally show a strong influence of σ_v on suction-water content relationships; however, a significant pressure effect could exist for soils taken from relatively shallow depths if pressures above the maximum past pressure are applied. The assumption is made that there is essentially no pressure effect on suction-water content relationships or no change in $\overline{A}$ and $\overline{B}$ parameters, except for the pressure effect defined by Equation 4, if applied pressures are less than the maximum past consolidation pressure. In other words, there will be no change in structure for pressures less than the maximum past pressure. A settlement due to compression may occur for applied pressures greater than the maximum past consolidation pressure.

Effect of field confining pressure

97. Field measurements of piezometric pressure at the Clinton, Fort Carson, and LAFB test sections (Figure 31) were used with suction calculations to determine rough values of the coefficients of earth pressure at rest K_o from Equations 7 and 10 (Table 11). Limits in the suction τ_{mo}^o were roughly estimated from observed variations in water content (Figures 17-20). The lower value between τ_{mo}^o and the suction swell pressure was generally taken for τ_{mo}^o . Many of these soils are below the water table and should have a degree of saturation of one in the field. Soils beneath these test sections are overconsolidated and coefficients greater than one are expected. Coefficients greater than one suggest that most or all volume changes will probably occur in the vertical direction such that the appropriate compressibility factor may be given by Equation 15.

98. The coefficients in Figure 32 contain considerable scatter and tend to be high, which may indicate some sample disturbance and sample drying. All of the data points are plotted in a single figure

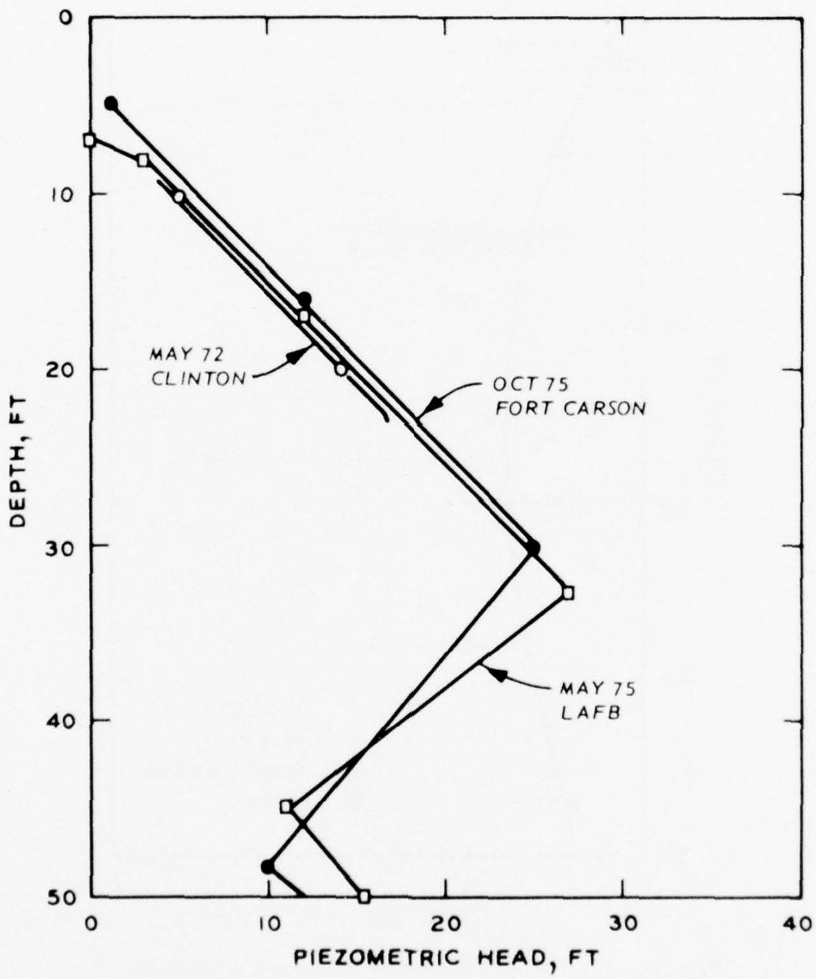

Figure 31. Piezometric pressure distribution with depth

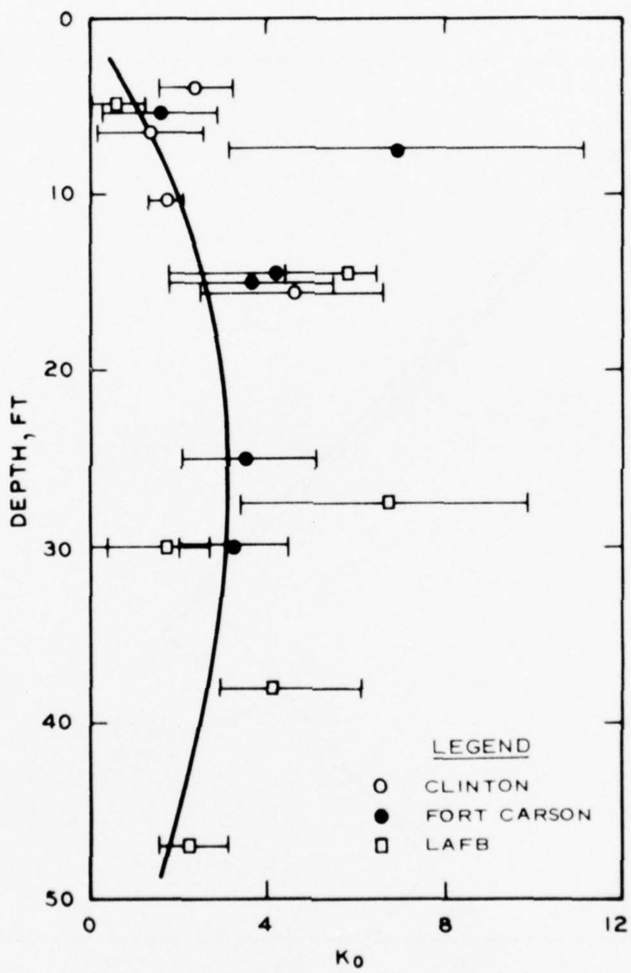

Figure 32. Coefficients of earth pressure as a function of depth

due to the limited data and observed scatter. The coefficients appear smaller near the ground surface at about 5 ft of depth and near the top of the water tables at these test sections. The coefficients reach a maximum near 30 ft of depth. K_o appears to be about one to two above 10 ft, increases to three at 20 to 30 ft, and decreases down to about two to three below 30 ft. $\bar{K}_o$ (Table 11) is greater than K_o for positive pore pressures. The existence of high lateral pressures (Figure 32) indicates that 1D swell tests, in which there is no control over lateral pressures, may not be appropriate tests for predicting in situ heave, especially for clay shales.

99. The effective coefficients of earth pressure at rest $\bar{K}_o$ (Table 11) can be compared with results of the Brooker and Ireland[55] data if the overconsolidation ratio (OCR) is known. The maximum past pressure was evaluated on four soils for which comparisons of $\bar{K}_o$ may be made:

Sample	Depth ft	σ_{vo} tsf	Maximum Past Pressure tsf	OCR	$\bar{K}_o$ Brooker and Ireland[55]	$\bar{K}_o$ Table 11
LAFB No. 1						
4	4.3-5.2	0.29	0.84	3	0.8	1.0 ± 0.5
17	29.0-30.0	1.85	14	8	1.2	2.1 ± 1.7
Fort Carson						
P1-5	4.0-5.25	0.29	0.67	2	0.7	1.7 ± 1.3*
BOQ3-23	29.5-30.5	1.88	39	21	2.0	5.0 ± 2.7

* Sample BOQ3-4, 5.7 - 7.0 ft of depth.

The results of $\bar{K}_o$ between the methods for the above four soils are similar. The experimental errors can be reduced in subsequent tests by analyzing numerous specimens resulting in more reliable water content, void ratio, and suction parameters.

Comparison of Volume Changes

Swell potential

100. The methods for evaluating the relative degree of expansion of soils by the Bureau of Reclamation (BR),[14] Dakshanamurthy and Raman (DR),[15] and Seed, Woodward, and Lundgren (SWL)[16] give reasonably

consistent ratings (Table 12). In addition to these methods, the suction method is suggested that is rated on the basis of the volumetric suction index

Degree of Expansion	$C_{\tau\alpha}$
Low	<0.04
Medium	0.05 to 0.10
High	0.11 to 0.20
Very High	>0.21

The above suction rating is also generally consistent with the three previous methods, except that the Fort Carson soils below 20 ft of depth (BOQ3-20 and deeper) are given only a medium rating by suction, while the other methods rate these soils with a high or very high swelling potential. The structure in the Pierre shale may have led to the smaller rating by the suction method, whereas the other methods give swelling potentials independent of structure. All of these methods can be used with about equal simplicity and economy; however, quantitative information is not provided and reliable indications of in situ heave cannot be expected.

101. Quantitative comparisons of swell potentials from the initial water content to saturation under a surcharge of about 1 psi were made for the SWL,[16] Vijayvergiya and Sullivan (VS),[17] Nayak and Christensen (NC),[18] and Vijayvergiya and Ghazzaly (VG)[19] methods. Calculations (Table 12) show that all these methods are generally consistent in indicating relative degrees of expansion or magnitudes of volume change. The SWL and NC methods usually give significantly larger swell potentials than the VS and VG methods. The SWL procedure more often gives greater potentials than the other methods. The VG method leads to somewhat smaller swell potentials than the VS method, which is consistent with the slightly larger confining pressure of 1.5 psi (0.1 tsf). Some swell potentials calculated by the VS and VG methods are blown out of reasonable proportions with respect to the other methods for the more dense Fort Carson samples below 20 ft of depth (BOQ3-20 and deeper).

Volume changes

102. Comparisons were made between the percent volume changes

calculated by different methods described in Table 3. Final field conditions assumed were saturation ($\tau^o_{mf} = \sigma_{vo}$) under the estimated field confining pressure σ_{vo} . These volume changes correspond to the general definition of swell potential given in paragraph 23. The calculation of volume changes by the suction method was made from Equation 22 assuming initial suctions τ^o_{mo} given in Table 13 and a compressibility factor α from Equation 15. The final suction was taken as equal to the confining pressure from Equation 6 where σ_v is σ_{vo} . The two cases of K_o equal to one and K_o from Figure 32 were also assumed in the suction method calculations (Table 13).

103. Field test sections. Soils beneath the field test sections at Clinton, LAFB (test pier site), and Fort Carson all contain perched water tables (Figure 31). These soils may also have some lateral pressures (Figure 32). The calculated volumetric changes for depths within the water tables should be negative assuming a final moisture profile of zero in situ pore water pressure or $\tau^o_{mf} = \sigma_{vo}$. In contrast, field measurements of heave at the Clinton test section were about 1 in. by October 1975 after 7 years.[57] Heave of about 1 in. was observed November 1975 beneath the test section at LAFB following construction in July 1973. Insignificant heave had accumulated beneath the test section at Fort Carson following construction in November 1973.

104. Percent volume changes beneath the field test sections as a function of depth are shown in Figures 33-35 (Table 13). Most calculated volume changes show similar (positive) swells between methods, except for the apparently excessive volume changes computed by the water content/plastic limit (w/PL) ratio[34,35] from[10]

$$\frac{\Delta V}{V} = \frac{\Delta w G_s}{100 + w_o G_s} \qquad (41)$$

The suction method with K_o of one usually provides somewhat larger volume changes than the other methods, except for the w/PL ratio method. The suction method with K_o from Figure 32 gives smaller volume changes relative to K_o of one, with some soils indicating a decrease in volume. The four laboratory swell tests performed with the ISO procedure

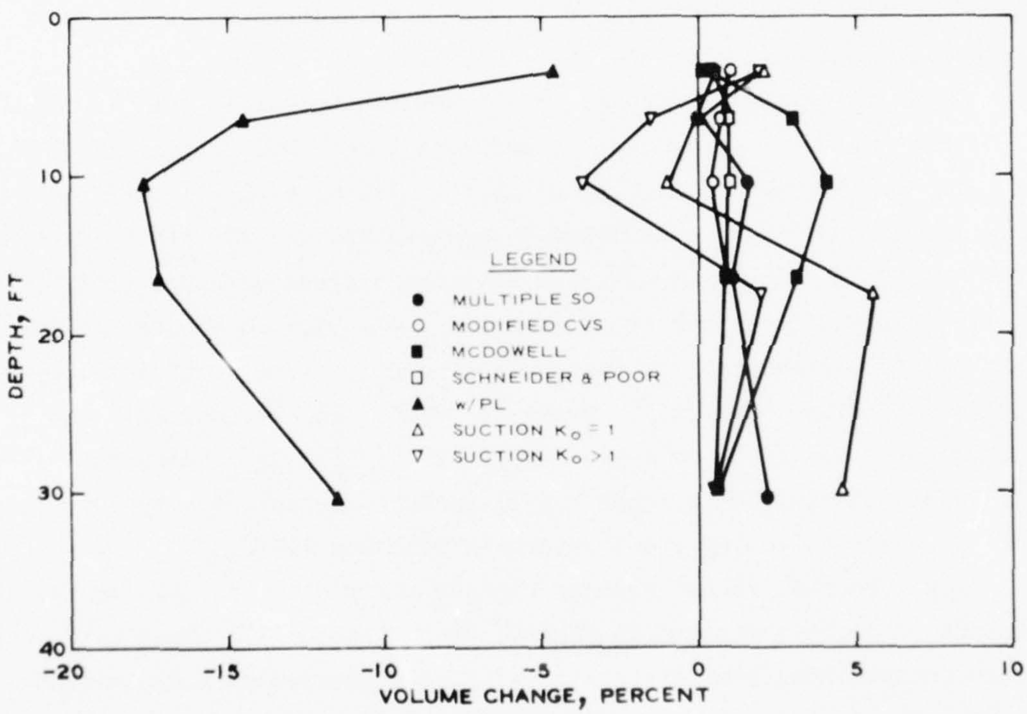

Figure 33. Comparison of volume changes for soils from the Clinton test section

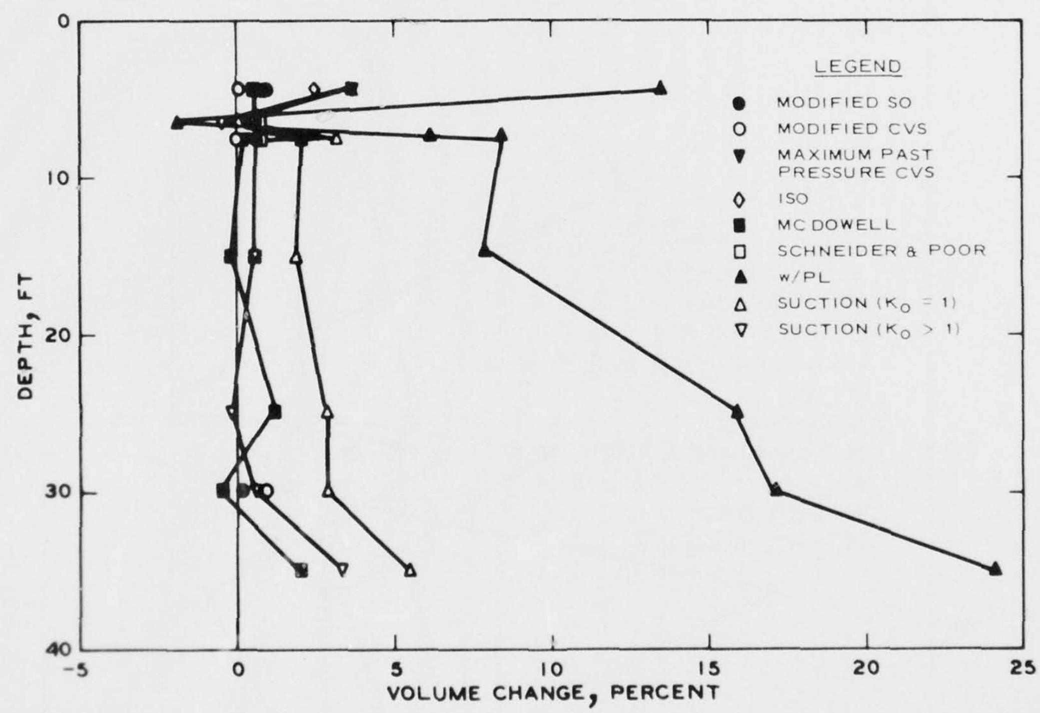

Figure 34. Comparison of volume changes for soils from the Fort Carson test section

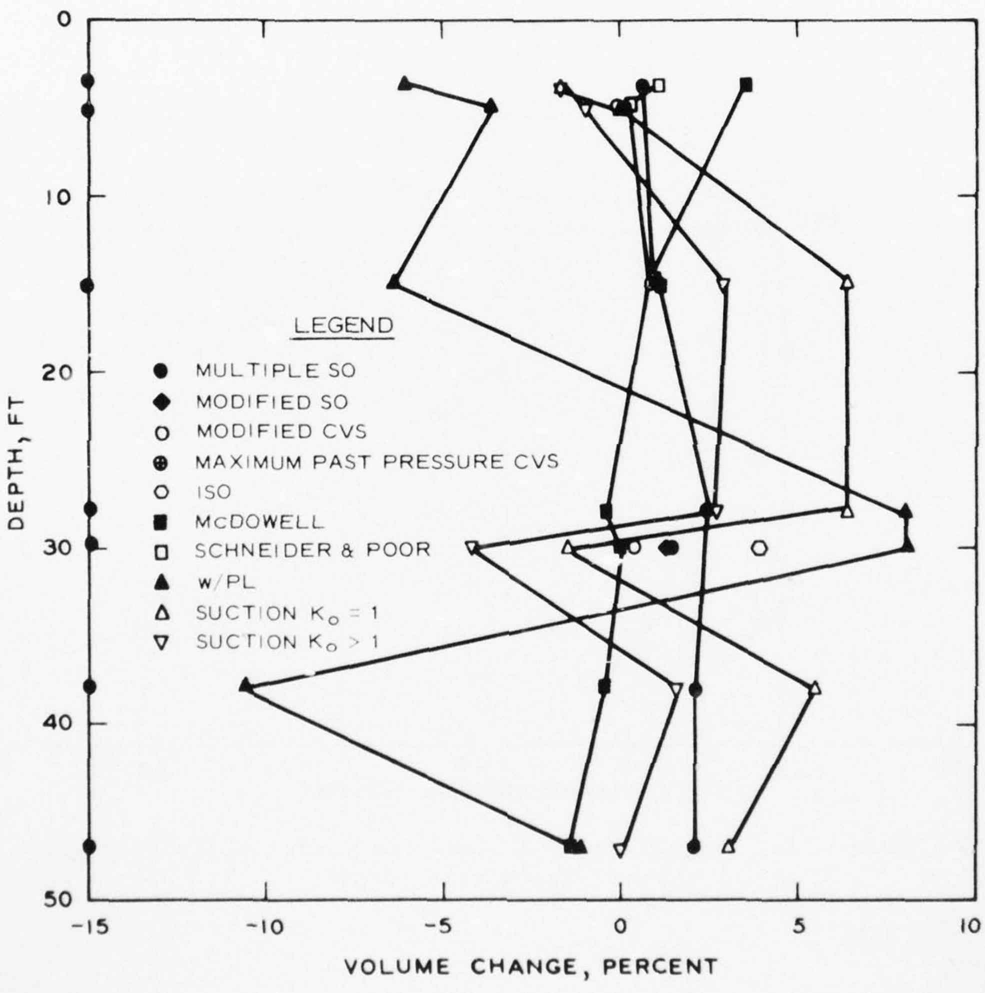

Figure 35. Comparison of volume changes for soils from LAFB No. 1 (test pier site)

led to larger swells than the other CS tests, but not enough tests were performed to statistically evaluate the relative difference in magnitude between these methods.

105. The laboratory swell tests apparently do not adequately consider lateral pressures because computed volume changes are not negative as should be the case assuming final zero pore water pressures. Results from the McDowell[25] and Schneider and Poor[36] methods are similar to the results of the laboratory swell tests. The lateral pressures taken for the Fort Carson and LAFB soils may not be assumed high enough since the suction method with K_o from Figure 32 still suggests some positive volume changes. The soil samples may also not be completely representative of the initial in situ soil conditions due to sample disturbance, sample drying, and heterogeneous soils.

106. The w/PL concept, which is supposed to provide an equilibrium field water content, is too rigid and not capable of considering variations in field conditions that may lead to differences in availability of water and surcharge pressures. Figures 33-35 indicate that the w/PL concept is probably not applicable to these soils.

107. Other testing areas. Figures 36 and 37 show comparisons of volume changes for soils from LAFB, Fort Sam Houston, and KAFB. Volume changes from laboratory swell tests are again similar to volume changes from the McDowell[25] and the Schneider and Poor[36] methods. The w/PL ratio again indicates apparently excessive volume changes. The suction method also indicated some volume changes larger than those from laboratory swell tests. The negative volume changes calculated by the suction method for KAFB soils below 8 ft of depth show the existence of a water table. Groundwater is known to occur in different areas of KAFB.

108. Swells measured from CS tests may not be as large as swells calculated by the suction method because the time allotted for swell may not be adequate to achieve full swelling. Jennings et al.[27] and Prendergast et al.[58] indicate that laboratory swell tests may underpredict heave. Jennings et al. found that predictions of in situ heave were about one half of the actual heave if results were used from SO tests. The suction and ISO methods may lead to better estimates of in

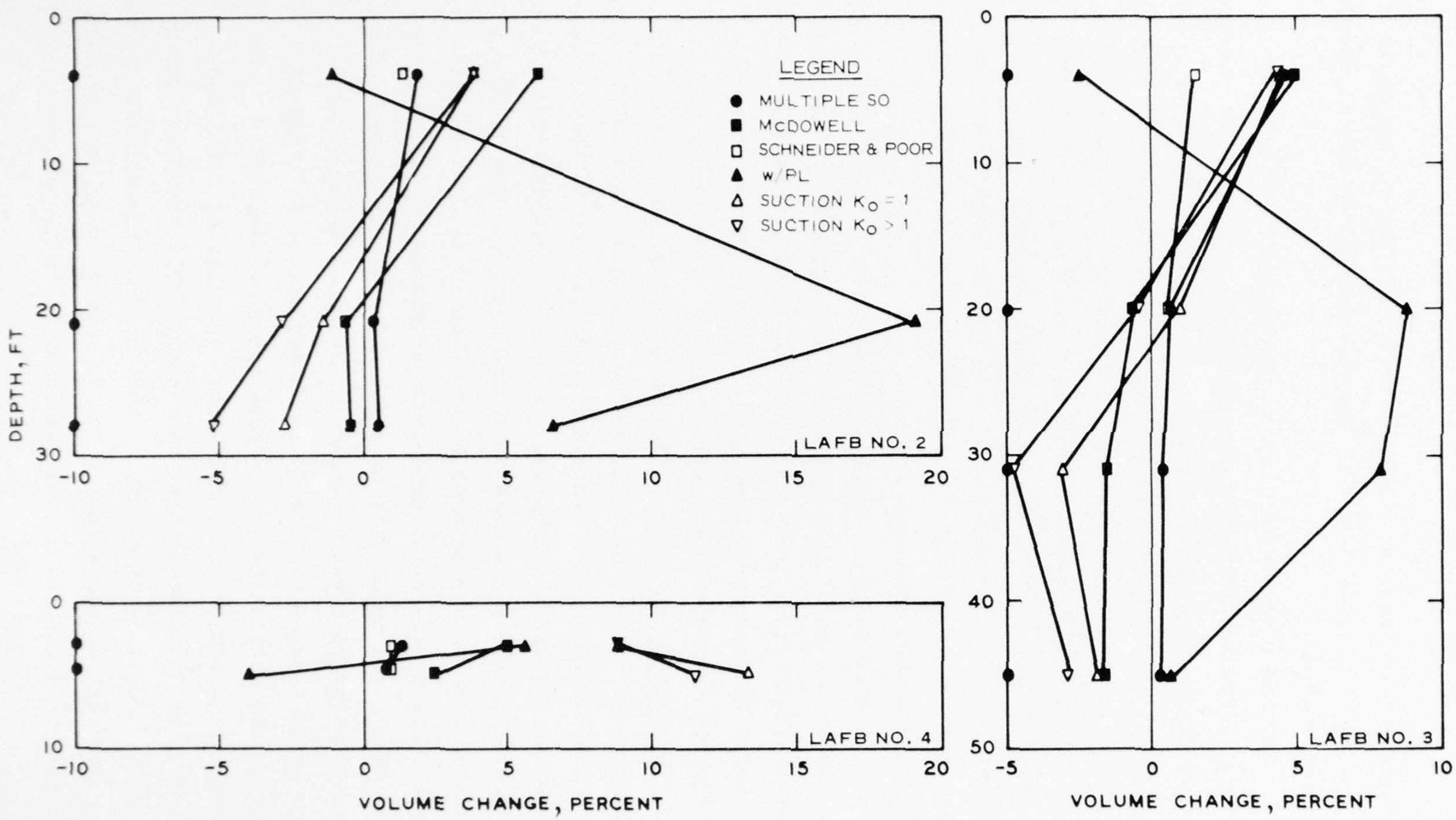

Figure 36. Comparison of volume changes for soils from LAFB samples Nos. 2, 3, and 4

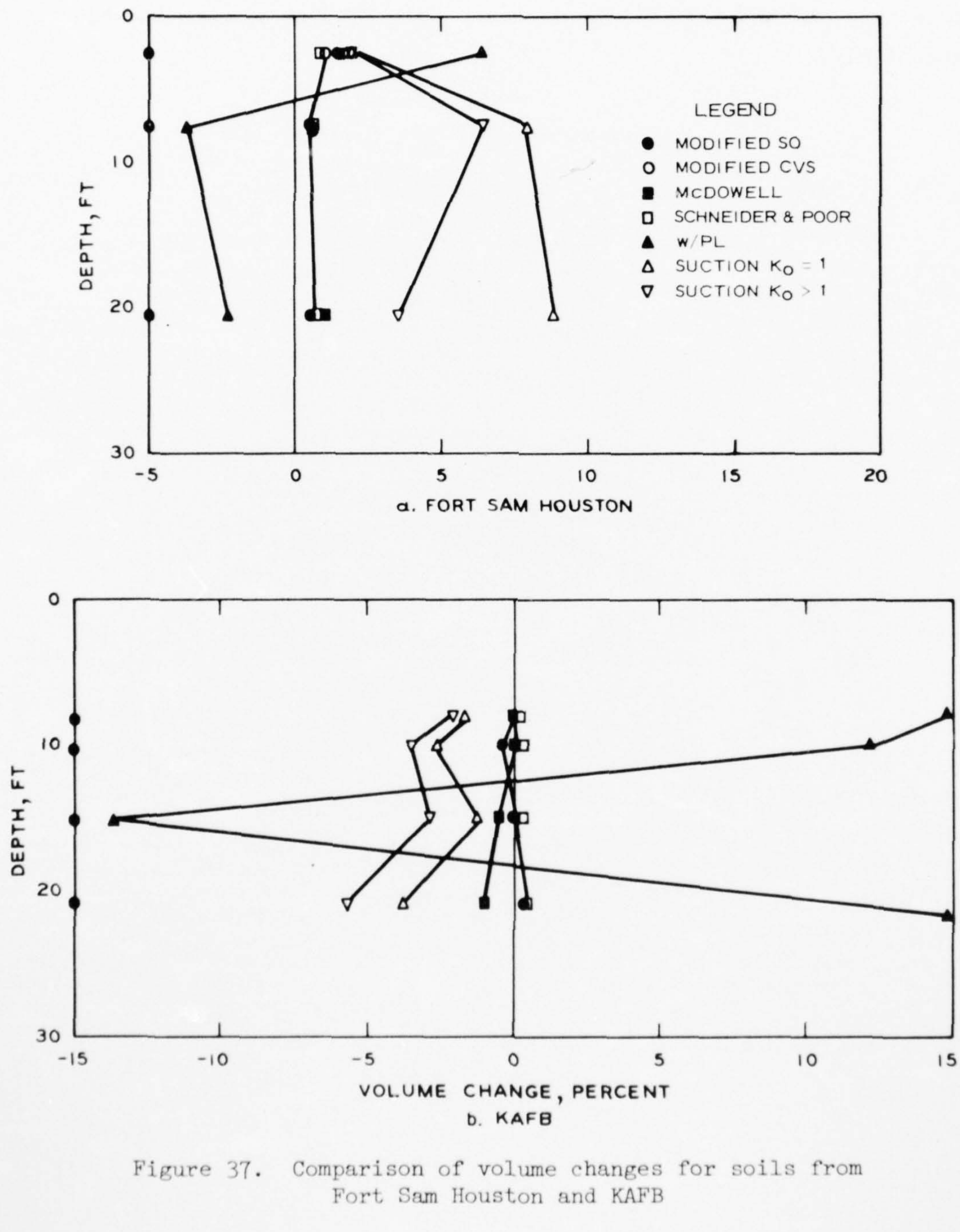

Figure 37. Comparison of volume changes for soils from Fort Sam Houston and KAFB

situ heave if final field conditions, such as vertical and lateral pressures and the equilibrium moisture profile, can be approximated adequately.

PART V: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

109. Methods of determining swell potential often use easy to obtain soil classification data, which provide fairly consistent values for relative degrees of expansion; however, swell potential is not a reliable indicator of in situ heave. Greater potentials for swell are commonly indicated by larger LL's and PI's without regard to the effect of structure on swell. The suction method developed herein shows much promise as a useful technique for rating swell potential while including the effects of structure.

110. Predictions of in situ heave are commonly made from results of swell tests performed in the 1D consolidation apparatus. These tests are time-consuming and can be difficult to perform. Results from consolidation swell tests may be unreliable if time is not adequate to achieve full saturation and swell. Lateral pressures, which can be significant in clay shales, are also ignored during these swell tests. Swell pressures and volume changes evaluated from results of different swell tests show considerable scatter that is probably attributable to heterogeneous soil samples, sample disturbance, and the different testing procedures.

111. The suction method of determining swell pressures and volume changes from soil suction data is simple and easy to perform, takes little time, and requires relatively inexpensive equipment compared to many other methods. The suction method appears to simulate field conditions as well as or better than other methods that use data from laboratory swell tests and, therefore, the suction method can be applied for predicting in situ heave.

112. Overall, comparisons of swell pressures calculated by the suction method are similar to results from laboratory swell tests. The extent of the correlations of suction swell pressures with swell pressures measured from laboratory swell tests depends strongly on the method of testing. Suction swell pressures appear to correlate better

with swell pressures measured from modified SO and CVS, maximum past pressure CVS, and ISO test results. The suction swell pressure is found to be a function of the composition, structure, void ratio, and specific gravity.

113. Theory of the suction method shows that soil suction measured under zero confining pressure converges to effective stress when the degree of saturation is one. The equation for predicting volume changes from suction data is analogous to the consolidation equation for calculation of settlements. Suction indices computed by the suction method are larger than swell indices and smaller than compression indices measured from relatively short-term 1D consolidation swell tests. Volume changes predicted by the suction method may therefore tend to be larger for identical moisture and confining conditions than volume changes calculated by many other methods. Suction indices with the compressibility factor set equal to one appear to be close to the compression indices based on limited test data.

114. The effects of lateral pressures on heave can be significant and should be considered for overconsolidated soils. Volume changes predicted by various methods may be excessive if significant lateral pressures exist and if these pressures are not considered in the calculations.

Recommendations

115. Comparisons of heave predictions with field data are needed to fully evaluate different methods for predicting in situ heave. Extensive laboratory studies are needed to thoroughly investigate the more promising methods of determining swell pressure and volume changes such as the suction, modified SO and CVS, and maximum past pressure CVS methods used in making in situ predictions of heave. Swell tests such as the modified SO and CVS and maximum past pressure CVS tests need to be performed over extensive periods of time, perhaps many months, to ensure that all swell has occurred, for appropriate comparisons with other test procedures such as the suction and ISO methods. Simple and

economical methods, such as those of McDowell[25] and Schneider and Poor,[36] should also be used in comparison studies.

116. Research is needed to investigate the limitations of the suction method at low levels of suction for determining swell pressures and volume changes. A theory may be needed for determining swell pressures and volume changes at low suction levels and for describing the effects of osmotic suction on swell pressure and swell when water available to the soil has a different chemical composition than the pore water.

117. Research is needed to investigate relationships between soil suction, strength, and volume change behavior to understand the effect that soil strength has on suppressing swell pressure and volume changes. The compressibility factor for applied pressure α_σ and the volumetric compressibility α_s need study to understand differences between these factors and to determine simple methods of evaluation. The volumetric compressibility factor α_s may be easily evaluated on undisturbed pieces during the suction measurements. Simple ways of evaluating K_o are also needed to better estimate field confining pressures, which will increase confidence in heave predictions. Other recommendations include performing swell tests on soil samples retained in the original sampling liners and determining void ratios directly on specimens used to evaluate the $\bar{A}$ and $\bar{B}$ parameters, thus increasing reliability of results.

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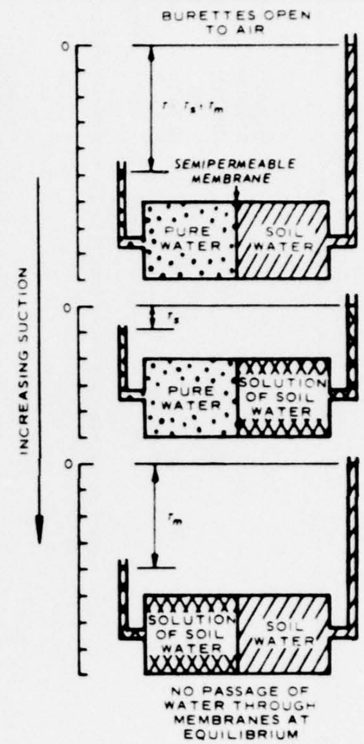
Table 1

Definitions of Suction '

Term	Symbol	Definition*	Illustration
Total suction	τ	The negative gage pressure, relative to the external gas pressure** on the soil water, to which a pool of pure water must be subjected in order to be in equilibrium through a semipermeable (permeable to water molecules only) membrane with the soil water	
Osmotic (solute) suction	τ_s	The negative gage pressure to which a pool of pure water must be subjected in order to be in equilibrium through a semipermeable membrane with a pool containing a solution identical in composition with the soil water	
Matrix (soil water) suction	τ_m	The negative gage pressure, relative to the external gas pressure** on the soil water, to which a solution identical in composition with the soil water must be subjected in order to be in equilibrium through a porous permeable wall with the soil water	



* From Reference 3 of text.

** The magnitude of the matrix suction is reduced by the magnitude of the external gas pressure. The osmotic suction is determined by the concentration of soluble salts in the pore water and can be given by $\tau_s = RT/V_{mv} \log_e P/P_0$ where R is the universal gas constant, T is absolute temperature, V_{mv} is volume of a mole of liquid water, P is vapor pressure of the pore water extract, and P_0 is vapor pressure of free pure water.

Table 2

Procedures for Determining Swell Potential

Method	Description

Bureau of Reclamation[14]

Colloid Content percent <1 μ	PI	SL	Probable Expansion percent	Degree of Expansion
<15	<18	>15	<10	Low
13 to 23	15 to 28	10 to 16	10 to 20	Medium
20 to 31	25 to 41	7 to 12	20 to 30	High
>28	>35	<11	>30	Very High

The expansion measured on wetting from air-dry to saturation under a 1-psi surcharge pressure

Dakshanamurthy and Raman[15]

LL	Degree of Expansion
20 to 35	Low
35 to 50	Medium
50 to 70	High
70 to 90	Very High
>90	Extra High

Seed, Woodward, and Lundgren[16]

PS	Degree of Expansion
<1.5	Low
1.5 to 5.0	Medium
5.0 to 25.0	High
>25.0	Very High

$PS = 0.00216PI^{2.44}$ compacted at optimum water content at maximum density to saturation for 1-psi surcharge pressure (useful for compacted soils)

Vijayvergiya and Sullivan[17]

$\log_{10} PS = 0.0526\gamma_d + 0.033LL - 6.8$ from natural water content for 1-psi surcharge pressure (useful for natural soils)

Nayak and Christensen[18]

$PS = 0.0229PI^{1.45}C/w_o + 6.38$ for 1-psi surcharge pressure (useful for compacted soil)

Vijayvergiya and Ghazzaly[19]

$\log_{10} PS = 1/12(0.4LL - w_o + 5.5)$ from natural water content w_o to saturation for a 0.1-tsf surcharge pressure (useful for natural soils)

PVC meter[20]

The potential volume change (PVC) based on a field testing device in which the swelling pressure under a 200-psf surcharge pressure at the end of 2 hr is related to a rating of the PVC meter. Swell pressures are underestimated due to swell that compresses the proving ring

Expansion index test[7]

A swell test performed in a 1D consolidation frame on a compacted specimen with degree of saturation adjusted between 49 and 51 percent. The volume change after submerging the specimen in distilled water is measured for a 1-psi surcharge pressure (useful as a basic swelling index property)

Dielectric dispersion test[21,22]

The difference in dielectric constant at frequencies of alternating current at about 3×10^6 Hz and 75×10^6 Hz is the magnitude of dielectric dispersion $\Delta\epsilon_o$. Mineralogy and amount of clay predominately affect $\Delta\epsilon_o$. $\Delta\epsilon_o$ is related to the expansion index determined by the expansion index test

Table 3

Procedures for Predicting Heave

Method	Description	Advantages	Disadvantages
Van der Merwe[24]	An empirical method that involves fitting field data of South African soils. Heave, feet, is computed by $H = \sum_{D=1}^{D=h} F \cdot (PE)_D$, where D = depth, ft; $(PE)_D$ = potential expansiveness per foot of depth; F = reduction factor to allow for surcharge (found by $D = f \log_{10} F$, where $f = 20$ for South African soils); $(PE)_D$ = 0, 1/4, 1/2, and 1 in. per foot of soil in South Africa for low, medium, high, and very high degrees of expansion, respectively. Degrees of expansion are determined from PI and clay fraction	No swell tests needed, quick, economical	Applicable only for local field conditions in South Africa. Good only for rough estimates of heave because variations in soil structure, availability of water to soil, and environment are not considered
McDowell (PVR)[25]	A popular procedure based on swell test results of many specimens of compacted Texas soils, usually applied in design of highway subgrades. The field heave is estimated from a family of curves using Atterberg limits, natural water content, and surcharge pressures for each soil stratum. The potential vertical rise (PVR) is one third the volume change of the soil profile	No swell tests needed, quick, economical	Applicable only for compacted soils and local field conditions. Good only for rough estimates of heave because variations in field conditions are not considered
Double oedometer[26]	One of two identically prepared undisturbed specimens is consolidated at natural water content in a 1D consolidometer. Distilled water is added to the other specimen under negligible pressure, and it is allowed to swell then consolidated using routine procedures. Virgin curves of the void ratio-log pressure relationship of the natural and soaked curves are adjusted to coincide. Heave is found from the difference in void ratio between the natural curve at the initial total pressure and the soaked curve at the final effective stress. Heave is assumed equal to the volumetric change	Considers composition, structure, initial moisture, and surcharge for useful prediction of heave depending on assumed final effective stress	Heave often overestimated. Correction technique is required if soil can collapse as well as heave. Degree of saturation at final moisture must be one
Improved simple oedometer[27]	One undisturbed specimen is subjected to original surcharge to find initial void ratio, the surcharge is removed, distilled water is added, and the specimen is allowed to swell then consolidated using routine procedures. The final void ratio is found from the soaked curve using the final effective stress based on the equilibrium suction. Heave is assumed equal to the volumetric change	Same as double oedometer method, but simpler and more economical	Degree of saturation at final moisture must be one
Swell overburden[28]	The vertical swell is measured on undisturbed soil of each stratum in a 1D consolidometer under total surcharge after wetting with distilled water. The volume change is the area bounded by the curve of the depth versus percent swell relationship. Final effective pressure is assumed to be the total surcharge. Heave is assumed equal to the volumetric change	Considers composition, structure, initial moisture, and surcharge	Degree of saturation at final moisture must be one. Heave is predicted for only one case of surcharge pressure. Swell pressures are not measured
Constant volume swell[29]	An undisturbed specimen from each stratum is loaded to the soil overburden pressure in a 1D consolidometer, distilled water is added, and the loading arm is restrained from movement until the full swelling pressure is developed. The specimen is unloaded incrementally allowing sufficient time at each increment to reach equilibrium. The final void ratio is found from the rebound curve and the final effective stress. Heave is assumed equal to the volumetric change	Same as swell overburden method	Degree of saturation at final moisture must be one
Sampson, Schuster, and Budge[30]	This method was applied to predictions of heave in highway cuts. Three identical undisturbed specimens are required for each stratum. One specimen is loaded to the maximum (32 tsf) and rebounded to a small value (0.1 tsf) in one increment to determine the swell index. A second specimen is loaded to the original surcharge and distilled water is added to obtain swell. The third specimen is loaded to the original surcharge, distilled water is added, and a portion of the load equivalent to the cut is removed. The volume change is found from the swell index and changes in the void ratio. The final effective pressure is assumed to be the surcharge after the cut is removed. Heave is assumed equal to volumetric change	Same as swell overburden method	Degree of saturation of final moisture must be one. Heave predicted for only one case of surcharge pressure
Noble[31]	Four statically compacted specimens with two different water contents are prepared. The specimens are permitted to swell on addition of distilled water under two different surcharge pressures. Previously correlated data are consulted to determine the volume change for changing loading and initial water content conditions	Swell pressure may be measured more consistently	Four swell tests instead of one needed for each stratum
Controlled suction[32]	A membrane oedometer may be used to apply soil suctions expected in the field. The change in volume can be recorded from changes in suction and loading conditions	The degree of saturation at equilibrium need not be one. This method considers composition, structure, initial moisture, and surcharge	Complex equipment is required. Test is time-consuming if a series of equilibrium suctions must be applied
Lytton and Watt[33]	Applies diffusion theory to solve 2D moisture flow by implicit finite difference technique. Heave is based on input of the volume-suction-pressure relationships. The availability of water can be input in terms of expected climatic conditions	Same as controlled suction method plus computes rate of heave	Procedure is complex and input data are difficult to obtain
Water content-plastic limit ratio[34,35]	The specimen is loaded to overburden pressure P_o plus a surcharge equal to the pore water suction expected in the field τ_m . τ_m is estimated from a plot of swell pressure versus water content/plastic limit (w/PL). The w/PL expected in the field is estimated as w/PL = 1.1 for PL = 20 ; w/PL = 1.2 for PL = 25 ; w/PL = 1.3 for PL = 35 to 40 . Water is added and swell is measured	Considers composition, structure, initial moisture, and surcharge	Degree of saturation at final moisture must be one. Swell pressure versus w/PL curve may be needed for each soil
Schneider and Poor[36]	Surcharge, ft: 0 — $\log_{10}$ Percent Swell = 0.90 (PI/w) - 1.19 3 — 0.65 (PI/w) - 0.93 5 — 0.51 (PI/w) - 0.76 10 — 0.41 (PI/w) - 0.69 20 — 0.33 (PI/w) - 0.62		

Table 4

Description of Soils

Location	Description of Soil	Climate
Clinton, Miss.	The primary soil, Yazoo clay, is a fat, stiff marine clay of the Jackson group. Upper portions of the Yazoo clay may be weathered to a yellow or light brown containing joints or slickensides. Gypsum crystals in the selenite form are commonly found. Lower portions of Yazoo clay are unweathered, not jointed, homogeneous, calcareous, fossilferous, blue-green to blue-gray color with some pyrite. Overburden soil is a lean clay, loessial material above about 8 ft below ground surface	Warm and humid, 50 in. of rain annually
LAFB, Tex. Test Pier Site (No. 1)	The primary soil, Upper Midway, is a weathered, tan, clayey shale. Overburden consists of about 8 ft of black, fat clay overlying about 4 ft of light, tan clayey shale	Semiarid, 30 in. of rain annually
Dental Clinic (Nos. 2, 3, 4)	The primary soil, Navarro, is weathered, sandy, calcareous, homogeneous shale with occasional limestone concretions. The light brown or gray shale is fractured, jointed, and contains some iron oxide. Above 18 ft it is a light-colored lean clay. A fat, black to brown clay lies above 6 to 8 ft.	
Fort Sam Houston, Tex.	The primary soil, Taylor, is a weathered, slightly silty, calcareous, tan clay shale. The shale is jointed and fractured to depths of more than 50 ft. Overburden averages 12 ft in depth of calcareous black, fat clay	Semiarid, 30 in. of rain annually
KAFB, Tex.	The primary soils, Lower Midway and Navarro, are sandy, tan clay shales. The soil is fractured, jointed, and weathered, containing large portions of white, calcareous deposits. The overburden consists of from 0 to 5 ft of black, fat clay	Semiarid, 30 in. of rain annually
Fort Carson, Colo.	The primary soil, Pierre shale, below about 8 ft is very stiff, sandy, dark gray, silty, carbonaceous claystone and siltstone containing frequent bentonite seams. Overburden is a brown, calcareous, fat, moist clay containing some gypsum	Semiarid, 23 in. of rain annually

Table 5

Classification Data

Sample	Depth, ft	Specific Gravity G_s	Atterberg Limits LL	PL	PI	Natural Water Content, percent	Natural Void Ratio	Clay Fraction C, Percent <2 μ
Clinton								
3	3.5 to 4.9	2.70	42	21	21	26.0	0.73	23
4	6.0 to 7.0	2.70	68	20	48	32.0	0.85	58
7	10.1 to 11.1	2.78	97	25	72	44.5	1.18	69
12	16.1 to 17.1	2.73	111	29	82	49.7	1.30	80
25	30.1 to 31.2	2.73	100	30	70	45.5	1.15	70
Fort Carson								
P1-5	4.0 to 5.3	2.75	43	22	21	17.0	0.53	28
BOQ3-4	5.7 to 7.0	2.75	49	19	30	22.0	0.60	37
P4-7	7.0 to 8.1	2.76	43	21	22	17.1	0.52	31
P4-9	14.6 to 15.7	2.78	43	21	22	15.0	0.52	31
BOQ3-10	14.7 to 15.7	2.75	43	22	21	17.3	0.42	41
BOQ3-20	24.7 to 26.0	2.76	70	19	51	13.0	0.37	46
BOQ3-23	29.5 to 30.5	2.74	54	17	37	10.6	0.31	40
BOQ3-27	34.2 to 35.2	2.71	73	19	54	9.6	0.25	35
LAFB No. 1								
3	3.2 to 4.2	--	69	23	46	31.7	0.97	42
4	4.3 to 5.2	2.78	58	26	32	33.8	1.04	62
11	14.3 to 15.3	2.76	73	23	50	31.9	0.87	53
15	27.3 to 28.3	2.75	78	30	48	30.6	0.83	51
17	29.0 to 30.0	2.76	80	29	51	29.4	0.87	55
23	37.4 to 38.7	2.73	82	21	61	30.2	0.81	57
28	46.5 to 47.4	2.73	74	24	50	29.6	0.77	52
LAFB No. 2								
4	3.6 to 4.3	2.68	74	24	50	29.5	0.78	34
9	20.4 to 21.5	2.72	50	25	25	19.2	0.52	28
11	27.7 to 28.6	2.71	54	22	32	20.5	0.55	32
LAFB No. 3								
2	3.2 to 4.6	2.70	67	22	45	25.7	0.76	--
18	19.8 to 20.8	2.71	45	21	24	18.2	0.44	28
29	31.1 to 32.4	2.72	54	22	32	19.6	0.54	32
40	44.7 to 45.8	2.72	49	18	31	19.4	0.51	33
LAFB No. 4								
2	2.8 to 3.9	2.69	75	29	46	31.6	0.88	--
3	4.0 to 5.0	2.71	67	24	43	31.0	0.94	--
Fort Sam Houston								
3	2.0 to 3.0	2.67	57	25	32	26.0	0.87	31
7	7.2 to 7.9	2.75	61	23	38	30.0	0.89	47
13	19.7 to 21.0	2.72	67	22	45	25.6	0.71	47
KAFB								
4	8.2 to 9.5	2.81	42	19	23	27.2 to 37.9	0.96	18
5	9.8 to 10.8	2.84	--	--	--	22.7 to 28.7	0.90	18
9	14.4 to 15.5	2.74	55	35	20	33.7 to 35.6	1.02	33
14	20.7 to 21.2	2.75	46	20	26	37.5 to 43.9	1.27	37

Table 6

Soil Suction Parameters

Sample	$\overline{A}$	$\overline{B}$	$\overline{A}$ - $\overline{B}$PL	$\overline{A} \div \overline{B}$
Clinton				
3	3.120	0.130	0.39	24.0
4	3.670	0.130	1.07	28.2
7	4.100	0.100	1.60	41.0
12	5.280	0.100	2.38	52.8
25	5.640	0.110	2.34	51.3
Fort Carson				
P1-5	3.766	0.182	-0.24	20.7
BOQ3-4	6.380	0.310	0.50	20.6
P4-7	3.766	0.182	-0.05	20.7
P4-9	3.766	0.182	-0.05	20.7
BOQ3-10	4.250	0.250	-1.00	17.0
BOQ3-20	3.850	0.250	-0.90	15.4
BOQ3-23	4.028	0.294	-0.97	13.7
BOQ3-27	5.210	0.410	-2.58	12.7
LAFB No. 1				
3	6.750	0.250	1.00	27.0
4	4.520	0.135	1.04	33.4
11	4.680	0.130	1.69	36.1
15	5.460	0.150	0.96	36.4
17	5.835	0.179	0.64	32.6
23	4.430	0.120	1.91	37.0
28	4.240	0.120	1.36	35.4
LAFB No. 2				
4	5.250	0.187	0.77	28.1
9	4.500	0.250	-1.75	18.0
11	3.920	0.200	-0.48	19.6
LAFB No. 3				
2	7.050	0.250	1.25	28.2
18	5.370	0.278	-0.46	19.3
29	5.170	0.278	-0.95	18.6
40	5.330	0.271	0.35	19.6
LAFB No. 4				
2	5.100	0.150	0.75	34.0
3	3.880	0.100	1.48	38.8

(Continued)

Table 6 (Concluded)

Sample	$\overline{A}$	$\overline{B}$	$\overline{A}$ - $\overline{B}$PL	$\overline{A} \div \overline{B}$
Fort Sam Houston				
3	4.780	0.170	0.53	28.1
7	3.920	0.111	1.36	35.3
13	3.56	0.111	1.11	32.0
KAFB				
4	6.25	0.303	0.49	20.6
5	5.02	0.227	0.70	22.1
9	3.81	0.111	-0.08	34.3
14	5.72	0.139	2.94	41.2

Table 7

Experimental Compressibility Factors

Sample	Depth, ft	PI	Vertical Compressibility Factor α
Clinton*			
4	3.5 to 4.3	17	0.04
10	11.0 to 12.1	88	0.93
18	21.0 to 22.1	70	0.67
26	31.0 to 32.1	73	0.64
LAFB*			
3	2.5 to 3.6	44	0.51
6	6.1 to 7.1	39	0.41
12	17.0 to 17.6	58	0.58
36	46.9 to 48.4	58	0.65
Fort Carson			
BOQ3-4	5.7 to 7.0	30	0.22
BOQ3-10	14.7 to 15.7	21	0.30
BOQ3-20	24.7 to 26.0	51	0.44
BOQ3-27	34.2 to 35.2	54	0.53

* From Reference 41.

Table 8

Comparisons of Initial Suction and Suction Swell Pressure

Sample	Initial Suction τ^{o}_{mo} , tsf	Suction Swell Pressure SP_s , tsf
Clinton		
3	0.58	0.43
4	0.34	0.40
7	0.47	0.76
12	2.16	3.49
25	4.57	10.73
Fort Carson		
P1-5	4.97	1.92
BOQ3-4	0.38	0.44
P4-7	4.77	2.30
P4-9	11.49	2.43
BOQ3-10	0.89	2.86
BOQ3-20	4.21	3.33
BOQ3-23	8.63	5.32
BOQ3-27	19.00	--
LAFB No. 1		
3	0.07	0.01
4	0.96	0.31
11	3.61	4.04
15	7.84	9.06
17	3.95	1.65
23	6.77	7.84
28	5.16	7.58
LAFB No. 2		
4	0.57	0.68
9	0.53	0.56
11	0.70	0.77
LAFB No. 3		
2	4.46	1.09
18	2.16	7.60
29	0.56	0.47
40	1.25	1.88
LAFB No. 4		
2	2.42	1.65
3	6.38	2.73

(Continued)

Table 8 (Concluded)

Sample	Initial Suction τ^{o}_{mo}, tsf	Suction Swell Pressure SP_s, tsf
Fort Sam Houston		
3	2.42	0.18
7	4.12	2.25
13	5.53	4.86
KAFB		
4	0.01	0.00
5	0.78	0.01
9	0.89	0.50
14	1.11	0.21

Table 9

Comparisons of Swell Pressures

Sample	Swell Pressure SP, tsf, in Cited Test						
	Suction	Multiple SO	Modified SO	Modified CVS	Maximum Past Pressure CVS	ISO	SP
Clinton							
3	0.43	1.50	--	1.15*	--	--	--
4	0.34	0.32	--	1.15*	--	--	--
7	0.47	3.05	--	0.95*	--	--	--
12	2.16	3.05	--	1.53*	--	--	--
25	4.57	14.80	--	2.62*	--	--	--
Fort Carson							
P1-5	1.92	--	1.00	0.41	0.79	1.10	0.26
P4-7	2.30	--	1.05	0.45	--	--	--
P4-9	2.43	--	0.94	1.46	--	--	--
BOQ3-23	5.32	--	3.20	2.40	7.20	4.61	0.26
LAFB No. 1							
3	0.01	1.20	--	--	--	--	--
4	0.31	--	0.40	0.30	0.46	0.39	0.14
11	3.61	6.70	--	--	--	--	--
15	7.84	33.10	--	--	--	--	--
17	1.65	--	6.40	2.85	5.10	8.00	0.87
23	6.77	33.10	--	--	--	--	--
28	5.16	33.10	--	--	--	--	--
LAFB No. 2							
4	0.57	11.30	--	--	--	--	--
9	0.53	3.00	--	--	--	--	--
11	0.70	5.85	--	--	--	--	--
LAFB No. 3							
2	1.09	3.23	--	--	--	--	--
18	2.16	4.40	--	--	--	--	--
29	0.47	10.70	--	--	--	--	--
40	1.25	10.70	--	--	--	--	--
LAFB No. 4							
2	1.65	2.50	--	--	--	--	--
3	2.73	1.62	--	--	--	--	--
Fort Sam Houston							
3	0.18	--	--	0.53	--	--	--
7	2.25	--	1.00	0.88	--	--	--
13	4.86	--	3.25	--	--	--	--
KAFB							
4	0.00	--	<0.50	--	--	--	--
5	0.01	--	Collapsed	--	--	--	--
9	0.50	--	<0.86	--	--	--	--
14	0.21	--	1.67	1.38	--	--	--

* From Reference 51.

Table 10

Suction, Swell, and Compression Indices

Sample	Compressibility Factors		Suction Index (Equation 23)		Swell Index C_s for Cited Test						Compression Index C_c	
	α (Equation 15)	$\overline{\alpha}$ (Equation 35)	$C_{\tau\alpha}$	$C_{\tau\overline{\alpha}}$	Multiple SO	Modified SO	Modified CVS	Maximum Past Pressure CVS	ISO	SP	Maximum Past Pressure	ISO
Clinton												
3	0.45	0.17	0.093	0.035	0.006	--	0.025*	--	--	--	--	--
4	1.00	0.45	0.208	0.093	0.004	--	0.025*	--	--	--	--	--
7	1.00	0.70	0.278	0.195	0.051	--	0.040*	--	--	--	--	--
12	1.00	0.81	0.273	0.221	0.053	--	0.100*	--	--	--	--	--
25	1.00	0.68	0.248	0.168	0.051	--	0.060*	--	--	--	--	--
Fort Carson												
P1-5	0.45	0.17	0.068	0.026	--	0.027	0.015	0.021	0.030	0.018	0.100	0.067
BOQ3-4	0.70	0.27	0.062	0.024	--	--	--	--	--	--	--	--
P4-7	0.47	0.18	0.071	0.027	--	0.010	0.014	--	--	--	--	--
P4-9	0.47	0.18	0.071	0.027	--	0.018	0.021	--	--	--	--	--
BOQ3-10	0.45	0.17	0.050	0.019	--	--	--	--	--	--	--	--
BOQ3-20	1.00	0.49	0.110	0.054	--	--	--	--	--	--	--	--
BOQ3-23	0.90	0.34	0.084	0.032	--	0.007	0.010	0.013	0.017	0.004	0.050	0.030
BOQ3-27	1.00	0.52	0.066	0.034	--	--	--	--	--	--	--	--
LAFB No. 1												
3	1.00	0.43	0.110	0.047	0.020	--	--	--	--	--	--	--
4	0.76	0.29	0.156	0.060	--	0.017	0.015	0.024	0.044	0.013	0.260	0.070
11	1.00	0.48	0.212	0.062	0.026	--	--	--	--	--	--	--
15	1.00	0.45	0.183	0.082	0.035	--	--	--	--	--	--	--
17	1.00	0.49	0.154	0.075	--	0.049	0.052	0.065	0.087	0.046	0.200	0.120
23	1.00	0.59	0.228	0.134	0.034	--	--	--	--	--	--	--
28	1.00	0.48	0.228	0.109	0.033	--	--	--	--	--	--	--
LAFB No. 2												
4	1.00	0.48	0.143	0.069	0.019	--	--	--	--	--	--	--
9	0.56	0.21	0.061	0.023	0.011	--	--	--	--	--	--	--
11	0.76	0.29	0.099	0.039	0.015	--	--	--	--	--	--	--
LAFB No. 3												
2	1.00	0.42	0.108	0.045	0.074	--	--	--	--	--	--	--
18	0.54	0.20	0.053	0.019	0.016	--	--	--	--	--	--	--

(Continued)

* From Reference 56.

Table 10 (Concluded)

Sample	Compressibility Factors		Suction Index (Equation 23)		Swell Index C_s for Cited Test						Compression Index C_c	
	α (Equation 15)	$\overline{\alpha}$ (Equation 35)	$C_{\tau\alpha}$	$C_{\tau\overline{\alpha}}$	Multiple SO	Modified SO	Modified CVS	Maximum Past Pressure CVS	ISO	SP	Maximum Past Pressure	ISO
LAFB No. 4												
2	1.00	0.43	0.179	0.077	0.022	--	--	--	--	--	--	--
3	1.00	0.40	0.271	0.108	0.022	--	--	--	--	--	--	--
Fort Sam Houston												
3	0.76	0.29	0.119	0.045	--	--	0.034	--	--	--	--	--
7	0.92	0.35	0.228	0.087	--	0.022	0.039	--	--	--	--	--
13	1.00	0.423	0.245	0.107	--	0.062	--	--	--	--	--	--
KAFB												
4	0.50	0.19	0.047	0.018	--	0.009	--	--	--	--	--	--
5	0.50	0.19	0.063	0.024	--	0.014	--	--	--	--	--	--
9	0.42	0.16	0.104	0.039	--	0.012	--	--	--	--	--	--
14	0.58	0.22	0.115	0.044	--	0.042	0.059	--	--	--	--	--

Table 11

Coefficients of Earth Pressure

Sample	Depth, ft	τ_m^o, tsf	u_w, tsf	σ_v, tsf	K_o (Equation 7)	$\overline{K}_o$ (Equation 10)
Clinton						
3	3.5 to 4.9	0.4 ± 0.1	0.0	0.2	2.5 ± 0.8	2.5 ± 0.8
4	6.0 to 7.0	0.4 ± 0.3	0.1	0.4	1.4 ± 1.2	1.5 ± 1.2
7	10.1 to 11.1	0.7 ± 0.2	0.2	0.6	1.7 ± 0.4	2.1 ± 0.4
12	16.1 to 17.1	3.0 ± 1.2	0.3	1.0	4.5 ± 2.0	5.9 ± 2.6
Fort Carson						
BOQ3-4	5.7 to 7.0	0.4 ± 0.3	0.1	0.4	1.6 ± 1.3	1.7 ± 1.3
P4-7	7.0 to 8.1	2.3 ± 1.3	0.2	0.5	7.0 ± 4.1	11.0 ± 6.3
P4-9	14.6 to 15.7	2.4 ± 1.4	0.4	0.9	4.2 ± 2.4	6.7 ± 4.2
BOQ3-10	14.7 to 15.7	2.4 ± 1.5	0.4	0.9	3.7 ± 2.5	6.7 ± 4.5
BOQ3-20	24.7 to 26.0	3.4 ± 1.5	0.7	1.5	3.6 ± 1.5	5.9 ± 2.8
BOQ3-23	29.5 to 30.5	4.0 ± 2.0	0.8	1.9	3.3 ± 1.6	5.0 ± 2.7
LAFB No. 1						
4	4.3 to 5.2	0.3 ± 0.1	0.0	0.3	1.0 ± 0.5	1.0 ± 0.5
11	14.3 to 15.3	4.0 - 0.5	0.3	1.0	5.9 ± 1.2	8.1 ± 1.1
15	27.3 to 28.3	8.0 ± 3.7	0.7	1.8	6.8 ± 3.0	10.4 ± 5.1
17	29.0 to 30.0	2.1 ± 1.5	0.8	2.0	1.7 ± 1.1	2.1 ± 1.7
23	37.4 to 38.5	7.0 ± 2.3	0.5	2.5	4.1 ± 1.4	4.8 ± 1.7
28	46.5 to 47.4	6.0 ± 2.3	0.3	3.0	2.7 ± 0.8	2.8 ± 1.2

Table 12

Comparisons of Swell Potential

Indicated by Cited Method

Sample	Degree of Expansion: Bureau of Reclamation[14]	Degree of Expansion: Dakshanamurthy and Raman[15]	Degree of Expansion: Seed, Woodward, and Lundgren[16]	Degree of Expansion: Suction	Volume Change for 1-psi Surcharge Pressure, percent: Seed, Woodward, and Lundgren[16]	Volume Change: Vijayvergiya and Sullivan[17]	Volume Change: Nayak and Christensen[18]	Volume Change: Vijayvergiya and Ghazzaly[19]*
Clinton								
3	Medium	Medium	Medium	Medium	3.6	0.5	8.0	0.5
4	Very High	High	Very High	Very High	27.3	1.7	17.7	1.1
7	Very High	Extra High	Very High	Very High	73.5	3.9	23.8	0.8
12	Very High	Extra High	Very High	Very High	100.9	5.8	28.2	1.0
25	Very High	Extra High	Very High	Very High	68.6	4.7	23.0	1.0
Fort Carson								
P1-5	Medium	Medium	Medium	Medium	3.6	3.3	9.5	3.0
BOQ3-4	High	Medium	High	Medium	8.7	5.2	11.7	1.3
P4-7	Medium	Medium	Medium	Medium	4.1	3.8	12.5	2.9
P4-9	Medium	Medium	Medium	Medium	4.1	4.2	11.6	4.4
BOQ3-10	Medium	Medium	Medium	Medium	3.6	9.5	10.8	2.8
BOQ3-20	Very High	High	Very High	High	31.7	133.3	30.6	50.9
BOQ3-23	Very High	High	High	Medium	14.5	70.9	22.6	23.7
BOQ3-27	Very High	Very High	Very High	Medium	36.4	534.4	33.4	123.1
LAFB No. 1								
3	Very High	High	High	High	24.6	1.3	14.2	1.3
4	High	High	High	High	10.2	0.4	12.7	0.4
11	Very High	Very High	Very High	Very High	30.2	2.8	17.4	1.7
15	Very High	Very High	Very High	High	27.3	5.1	16.8	3.2
17	Very High	Very High	Very High	High	31.7	4.9	19.2	4.3
23	Very High	Very High	Very High	Very High	49.0	7.2	23.1	4.7
28	Very High	Very High	Very High	Very High	30.2	5.1	18.0	2.9
LAFB No. 2								
4	Very High	Very High	Very High	High	30.2	3.8	14.0	2.9
9	High	High	High	Medium	5.6	5.3	9.9	3.5
11	High	High	High	Medium	10.2	5.3	11.8	3.5
LAFB No. 3								
2	Very High	High	High	High	23.3	2.8	13.9	3.5
18	High	Medium	High	Medium	5.0	7.3	9.9	2.8
29	High	High	High	Medium	10.2	6.1	12.0	4.2
40	High	Medium	High	Medium	9.4	5.4	12.0	3.0

(Continued)

* 0.1 tsf surcharge pressure.

Table 12 (Concluded)

Sample	Degree of Expansion				Volume Change for 1-psi Surcharge Pressure, percent			
	Bureau of Reclamation[14]	Dakshanamurthy and Raman[15]	Seed, Woodward, and Lundgren[16]	Suction	Seed, Woodward, and Lundgren[16]	Vijayvergiya and Sullivan[17]	Nayak and Christensen[18]	Vijayvergiya and Ghazzaly[19]
LAFB No.4								
2	Very High	Very High	High	High	24.6	2.4	13.0	2.1
3	Very High	High	High	Very High	20.9	1.0	12.3	1.3
Fort Sam Houston								
3	High	High	High	High	10.2	0.6	--	1.5
7	Very High	High	High	Very High	15.5	1.0	--	1.0
13	Very High	High	High	Very High	23.3	4.3	16.9	3.6
KAFB								
4	Medium	Medium	Medium	Medium	4.3	0.2	7.4	0.05
5	Medium	Medium	Medium	Medium	4.3	0.3	7.7	0.3
9	Medium	Medium	Medium	High	3.2	0.3	8.0	0.2
14	High	Medium	High	High	6.1	0.05	8.6	0.02

Table 13

Comparisons of Volume Change

Sample	Surcharge Pressure σ_{vo}, tsf	Volume Change from Swell Tests for $\overline{K}_o$ of 1, percent: Multiple SO	Modified SO	Modified CVS	Maximum Past Pressure CVS	ISO	Volume Change from Classification Data: McDowell[25]	Schneider and Poor[36]	w/PL Ratio[33,34]	Suction Method (Equation 7): τ^o_{mo}, tsf	Volume Change for K_o of 1, percent	K_o	Volume Change for K_o, percent
Clinton													
3	0.23	0.6	--	1.0*	--	--	0.1	0.4	-4.7	0.5	2.2	1.0	2.2
4	0.38	0.0	--	0.7*	--	--	3.0	1.0	-14.5	0.4	0.0	1.5	-1.4
7	0.61	1.6	--	0.4*	--	--	4.1	0.9	-17.7	0.5	-1.0	2.0	-3.8
12	1.00	1.1	--	0.8*	--	--	3.2	0.9	-17.2	3.0	5.7	2.5	2.1
25	1.77	2.2	--	0.5*	--	--	0.6	--	-11.5	4.6	4.7	3.0	0.5
Fort Carson													
P1-5	0.29	--	0.9	0.1	0.6	2.5	0.6	0.7	13.5	1.9	3.6	1.0	3.6
BOQ3-4	0.40	--	--	--	--	--	0.5	0.8	-1.9	0.4	0.0	1.5	-0.5
P4-7	0.45	--	0.2	0.0	--	--	0.5	0.7	11.2	2.3	3.2	2.0	2.1
P4-9	0.85	--	0.1	0.2	--	--	0.0	0.8	15.9	2.4	2.1	2.5	0.6
BOQ3-10	0.90	--	--	--	--	--	-0.2	0.6	12.8	2.9	1.8	2.5	0.7
BOQ3-20	1.50	--	--	--	--	--	1.2	--	15.8	3.3	2.8	3.0	-0.2
BOQ3-23	1.88	--	0.1	0.1	0.8	0.8	-0.5	--	17.2	5.3	2.8	3.0	0.5
BOQ3-27	2.00	--	--	--	--	--	2.0	--	24.3	19.9	5.3	3.0	3.3
LAFB No. 1													
3	0.24	0.7	--	--	--	--	3.5	1.0	-6.1	0.1	-1.7	1.0	-1.7
4	0.29	--	0.1	0.0	0.2	0.4	0.6	0.5	-3.7	0.3	0.0	1.5	-1.0
11	1.00	1.1	--	--	--	--	0.8	0.9	-6.3	3.6	6.3	2.5	2.9
15	1.83	2.4	--	--	--	--	-0.5	--	8.1	7.8	6.4	3.0	2.7
17	2.00	--	1.3	0.4	1.4	4.0	0.0	--	8.2	1.7	-1.5	3.0	-4.2
23	2.50	2.1	--	--	--	--	-0.5	--	-10.6	6.8	5.5	2.5	1.7
28	3.00	2.0	--	--	--	--	-1.5	--	-1.2	5.2	3.1	2.0	0.2
LAFB No. 2													
4	0.24	1.8	--	--	--	--	6.0	1.4	-1.1	0.6	3.8	1.0	3.8
9	1.30	0.3	--	--	--	--	-0.6	--	19.2	0.6	-1.4	3.0	-2.8
11	1.80	0.5	--	--	--	--	-0.5	--	6.5	0.7	-2.7	3.0	-5.1
LAFB No. 3													
2	0.24	4.7	--	--	--	--	5.0	1.5	-2.4	1.1	4.5	1.0	4.5
18	1.20	0.6	--	--	--	--	-0.6	0.6	8.9	2.2	1.0	3.0	-0.4
29	2.10	0.4	--	--	--	--	-1.5	--	8.1	0.5	-3.0	3.0	-4.8
40	3.00	0.3	--	--	--	--	-1.7	--	0.7	1.3	-1.8	2.0	-2.8
LAFB No. 4													
2	0.20	1.3	--	--	--	--	5.0	1.0	5.6	1.7	8.8	1.0	8.8
3	0.30	0.8	--	--	--	--	2.5	0.9	-3.9	2.7	13.3	1.5	11.6
Fort Sam Houston													
3	0.12	--	--	1.2	--	--	1.6	0.7	6.3	0.2	1.9	1.0	1.9
7	0.45	--	0.4	0.6	--	--	0.5	0.7	-3.6	2.3	8.0	1.5	6.5
13	1.16	--	0.6	--	--	--	1.0	0.9	-2.2	4.9	8.8	3.0	3.5
KAFB													
4	0.50	--	0.0	--	--	--	0.0	0.4	23.1	0.1	-1.7	1.5	-2.0
5	0.55	-0.3	--	--	--	--	0.0	0.4	12.2	0.1	-2.6	2.0	-3.3
9	0.86	0.0	--	--	--	--	-0.5	0.4	-13.7	0.5	-1.3	2.5	-2.9
14	1.14	0.3	--	--	--	--	-1.0	0.4	27.3	0.2	-3.8	3.0	-5.7

* From Reference 56.

APPENDIX A: NOTATION

$\overline{A}$	Suction parameter
B	Skempton's pore pressure parameter
$\overline{B}$	Suction parameter
$\overline{B}_r$	Suction parameter of remolded soil
$\overline{B}_u$	Suction parameter of undisturbed soil
BR	Bureau of Reclamation[14] (method)
C	Clay fraction (colloid content), percent <1 μ
C_c	Compression index
C_s	Swell index
CS	Consolidation swell (test)
C_{sr}	Swell index of remolded soil
C_{SU}	Swell index of undisturbed soil
CVS	Constant volume swell (test)
C_τ	Suction index ($\partial e/\partial \log \tau_m^o$)
$C_{\tau r}$	Suction index of remolded soil
$C_{\tau u}$	Suction index of undisturbed soil
$C_{\tau\alpha}$	Suction index with respect to the volumetric compressibility factor α
$C_{\tau\overline{\alpha}}$	Suction index with respect to the vertical compressibility factor $\overline{\alpha}$
DO	Double oedometer (test)
DR	Dakshanamurthy and Raman[15] (method)
e	Void ratio
e_f	Final void ratio
e_o	Initial void ratio
e_s	Void ratio at the seating pressure of a consolidometer following access to free water
$e_{\sigma_{vo}}$	Void ratio at the overburden pressure σ_{vo} following access to free water
E_t	Microvolt output at t°C
E_{25}	Microvolt output at 25°C
G_s	Specific gravity
h	Vertical distance above the depth of the active zone h_A , ft

h_A	Depth of the active zone below ground surface, ft
H	Thickness of clay stratum, ft
ISO	Improved simple oedometer (test)
K_o	Coefficient of earth pressure at rest with respect to total pressure
$\bar{K}_o$	Coefficient of earth pressure at rest with respect to effective pressure
LL	Liquid limit
NC	Nayak and Christensen[18] (method)
OCR	Overconsolidation ratio
p	Pressure of water vapor, atm
p_o	Pressure of saturated water vapor, atm
P	Vapor pressure of the pore water extract
P_e	Extraction air pressure applied to specimen in pressure membrane apparatus, atm
P_o	Loading pressure applied to specimen in pressure membrane apparatus, atm
P_O	Vapor pressure of free pure water
$(PE)_D$	Potential expansiveness per foot of depth, in.
PI	Plasticity index
PL	Plastic limit
PS	Percent swell
PVC	Potential volume change
Q	An exponent
R	Universal gas constant (82.06 cc-atm/°K-mole)
S	Degree of saturation, fraction
SP_s	Suction swell pressure, atm or tsf
SL	Shrinkage limit
SO	Swell overburden (test)
SP	Swell pressure, tsf
SWL	Seed, Woodward, and Lundgren[16] (method)
t	Temperature, °C
T	Absolute temperature, °K
u_w	Pore water pressure, tsf
V	Volume, cu ft

V_T	Specific total volume $(1 + e)/G_s$
VG	Vijayvergiya and Ghazzaly[19] (method)
VS	Vijayvergiya and Sullivan[17] (method)
V_{mv}	Volume of a mole of liquid water (18.02 cc)
w	Water content, percent dry weight
w_o	Initial water content, percent dry weight
w_A	Water content at air entry, percent dry weight
α	Volumetric compressibility factor
α_o	Compressibility factor for volume changes at zero water content
α_r	Compressibility factor of remolded soil
α_s	Compressibility factor for volume changes
α_u	Compressibility factor of undisturbed soil
α_σ	Compressibility factor for applied pressure σ
$\bar{\alpha}$	Vertical compressibility factor
γ_d	Unit dry density, g/cc
γ_w	Unit weight of water, g/cc
Δ	Change in; e.g., $\Delta\sigma$
σ	Total applied confining pressure, tsf
σ_f	Final total confining pressure, tsf
σ_h	Total horizontal pressure, tsf
σ_v	Total vertical pressure, tsf
σ_{vo}	Pressure in a consolidometer simulating the original in situ vertical overburden pressure, tsf
$\bar{\sigma}$	Effective pressure, tsf
$\bar{\sigma}_h$	Effective horizontal pressure, tsf
$\bar{\sigma}_v$	Effective vertical pressure, tsf
τ	Total suction, atm
τ^o	Total suction without surcharge pressure, atm
τ_m	Matrix suction under total applied pressure σ, atm or tsf
τ_s	Osmotic suction, atm
τ_{mf}	Final matrix suction under final total applied pressure σ_f, atm or tsf
τ_{mh}	Negative matrix suction head at distance h above the depth of the active zone h_A, ft
τ_{mh_A}	Negative matrix suction head at the depth of the active zone h_A, ft

τ_m^o	Matrix suction without surcharge pressure, atm or tsf
τ_{mf}^o	Final matrix suction without surcharge pressure, atm or tsf
τ_{mo}^o	Initial matrix suction without surcharge pressure, atm or tsf
1D	One-dimensional (apparatus)

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1. Foundations. 2. Predictions. 3. Soil suction. 4. Soil swelling. I. U. S. Army. Corps of Engineers. (Series: U. S. Waterways Experiment Station, Vicksburg, Miss. Technical report S-77-7)
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